Appendix A

PRELIMINARY GEOTECHNICAL INVESTIGATION
SAN TOMAS BUSINESS PARK
Santa Clara, California

Harvest Properties
Emeryville, California

5 June 2008
Project No. 4783.01

Treadwell&Rollo
Environmental and Geotechnical Consultants

Treadwell&Rollo

5 June 2008
Project No. 4783.01

Mr. Awais Mughal
Harvest Properties, Inc.
6475 Christie Avenue, Suite 550
Emeryville, CA 94608

Subject: Preliminary Geotechnical Investigation
San Tomas Business Park
Santa Clara, California

Dear Mr. Mughal:

We are pleased to submit herewith our preliminary geotechnical investigation report for the proposed San Tomas Business Park project at the southwest corner of San Tomas Expressway and Central Expressway in Santa Clara, California. The summary included in this letter omits detailed findings, conclusions, and recommendations; therefore, anyone relying on the report should read it in its entirety.

The property is bound by Central Expressway, Walsh Avenue, San Tomas Expressway, and existing buildings to the north, south, east, and west, respectively; the north-south trending San Tomas Aquino Channel runs through the middle of the site, between the Condensa parcel on the west side and the San Tomas parcel on the east side. Currently, the Condensa parcel is occupied by a building, surface parking, and landscaping. The San Tomas parcel is occupied by ten bungalow-style buildings surrounded by surface parking and numerous trees.

We understand the project will proceed in three phases based on a plan designated as Phasing Option A. The majority of the existing improvements at the site will be demolished and removed during Phase 1; five bungalow buildings will remain on the north side of the San Tomas parcel until Phase 2 is started and one bungalow building (#2650) at the southwest corner of the San Tomas parcel will remain as part of this development. Proposed development for Phasing Option A includes:

- **Phase 1:**
  - San Tomas Parcel: One garage structure with 1 level above grade and 2 levels below grade and at-grade construction of one 7-story office structure.
  - Condensa Parcel: One garage structure with 5 levels above grade and 2 levels below grade
- **Phase 2:**
  - San Tomas Parcel: At-grade construction of one 7-story office structure, which will be interconnected with the 7-story office structure constructed during Phase 1. Additional surface parking will be provided along the east side and north of the office structures.
  - Condensa Parcel: No new construction is planned for this parcel during this phase
- **Phase 3:**
  - San Tomas Parcel: One garage structure with 5 levels above grade and 2 levels below grade
  - Condensa Parcel: At-grade construction of an 8-story office structure

Additional site development includes surface roadways and landscaping.

The results of our investigation indicate the site is underlain by alluvial deposits consisting predominantly of clay and sandy clay layers interbedded with sand and clayey sand layers to the maximum explored depth of about 100 feet. Groundwater was encountered at a depth of approximately 13 feet during our investigation, corresponding to Elevation 29 feet.

Our preliminary conclusion is that the proposed parking garages with two basement levels may be supported on mat foundations. Similarly, we conclude the proposed 7-story and 8-story office structures, which will be constructed at-grade, should be supported on driven 14-inch-square precast, prestressed, concrete piles.

The recommendations and information provided herein are preliminary and should not be used for final design. A detailed geotechnical investigation should be performed to provide final design geotechnical parameters.

We appreciate the opportunity of assisting you with the design of this project, and we look forward to working with you during the final investigation, design, and construction phases.

Sincerely yours,
TREADWELL & ROLLO, INC.

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Enclosure

PRELIMINARY GEOTECHNICAL INVESTIGATION
SAN TOMAS BUSINESS PARK
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Harvest Properties
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PRELIMINARY GEOTECHNICAL INVESTIGATION
SAN TOMAS BUSINESS PARK
Santa Clara, California

1.0 INTRODUCTION

This report presents the results of the preliminary geotechnical investigation performed by Treadwell & Rollo, Inc. for the proposed San Tomas Business Park development at the southwest corner of San Tomas Expressway and Central Expressway in Santa Clara, California.

The property is bound by Central Expressway, Walsh Avenue, San Tomas Expressway, and existing buildings to the north, south, east, and west, respectively; the north-south trending San Tomas Aquino Channel runs through the middle of the site, between the Condensa parcel on the west side and the San Tomas parcel on the east side. The site is relatively flat, with elevations ranging from about 40 to 43 feet[1]. Currently, the Condensa parcel is occupied by a building, surface parking, and landscaping. The San Tomas parcel is occupied by ten bungalow-style buildings surrounded by surface parking and numerous trees.

We understand the project will proceed in three phases based on a plan designated as Phasing Option A. The majority of the existing improvements at the site will be demolished and removed during Phase 1; five bungalow buildings will remain on the north side of the San Tomas parcel until Phase 2 is started and one bungalow building (#2650) at the southwest corner of the San Tomas parcel will remain as part of this development. Proposed development for Phasing Option A includes:

- **Phase 1:**
  - San Tomas Parcel: One garage structure with 1 level above grade and 2 levels below grade and at-grade construction of one 7-story office structure.
  - Condensa Parcel: One garage structure with 5 levels above grade and 2 levels below grade

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1 Elevations are based on a topographic survey by Kier & Wright, dated January 2006, and reference Mean Sea Level datum.

Phase 2:

- San Tomas Parcel: At-grade construction of one 7-story office structure, which will be interconnected with the 7-story office structure constructed during Phase 1. Additional surface parking will be provided along the east side and north of the office structures.
- Condensa Parcel: No new construction is planned for this parcel during this phase

– **Phase 3:**

- San Tomas Parcel: One garage structure with 5 levels above grade and 2 levels below grade
- Condensa Parcel: At-grade construction of an 8-story office structure

Additional site development includes surface roadways and landscaping.

2.0 SCOPE OF SERVICES

Our scope of services was outlined in our proposal dated 14 February 2008. The purpose of our preliminary investigation was to provide preliminary geotechnical conclusions and recommendations for initial planning of the proposed development. We used the results of our investigation and performed laboratory tests and engineering analyses to develop preliminary conclusions and recommendations regarding:

- soil and groundwater conditions at the site
- site seismicity and seismic hazards, including potential for fault rupture, ground shaking, liquefaction, lateral spreading and seismically-induced settlements
- appropriate foundation type(s)
- design criteria for the recommended foundation types, including lateral load resistance and uplift capacity
- subgrade preparation for slab-on-grade floors
- 2007 California Building Code (CBC) soil and seismic factors
- site preparation and grading, including criteria for fill quality and compaction
- criteria for excavation and shoring, including tiebacks
- lateral pressures for basement walls, including a design earthquake increment
- construction considerations

3.0 FIELD EXPLORATION

Prior to drilling, we contacted Underground Service Alert (USA) and obtained the necessary permits from the Santa Clara Valley Water District (SCVWD).

To explore the subsurface conditions at the site we drilled one boring and performed one Cone Penetration Test (CPT) on the Condensa parcel and drilled two borings and performed two CPTs on the San Tomas parcel. We performed an exploration point within the footprint of each of the proposed structures. Details of each aspect of our field investigation are discussed in the following sections.

3.1 Borings

The borings, designated B-1 through B-3, were drilled to a depth of approximately 50 feet at the locations shown on Figure 2. Pitcher Drilling Company of Palo Alto drilled the borings on 5 and 6 March 2008 using a truck-mounted rotary wash drill rig. Our field engineer logged the borings and obtained samples of the material encountered for visual classification and laboratory testing. Logs of the borings are presented in Appendix A as Figures A-1 through A-3. The soil encountered was classified in accordance with the classification chart shown on Figure A-4.

Soil samples were obtained using three different types of samplers: two split-barrel samplers and one thin-walled sampler. The sampler types are as follows:

- Sprague and Henwood (S&H) split-barrel sampler with a 3.0-inch outside diameter and 2.5-inch-inside diameter, lined with brass tubes with an inside diameter of 2.43 inches
- Standard Penetration Test (SPT) split-barrel sampler with a 2.0-inch-outside and 1.5-inch-inside diameter, without liners
- Shelby tube (ST) piston sampler with a 3.0-inch outside diameter, thin-walled tube.

The sampler type was chosen on the basis of soil type and desired sample quality for laboratory testing. In general, the S&H sampler was used to obtain samples in medium stiff to very stiff cohesive soil and the SPT sampler was used to evaluate the relative density of sandy soil. The Shelby tube piston sampler was used to obtain relatively undisturbed samples of medium stiff to stiff cohesive soil.

The S&H and SPT samplers were driven with a 140-pound automatic hammer falling about 30 inches. Where the SPT and S&H samplers were used, the blow counts required to drive the sampler the final 12 inches of an 18-inch drive were corrected using a factor of 1.2 and 0.7, respectively, based on a 70 percent efficient hammer, to approximate SPT N_{60} blow counts and are shown on the boring logs. The Shelby Tube sampler uses 30-inch long stainless steel tubes, which are pushed hydraulically into the soil; the pressure required to advance the sampler is shown on the logs, measured in pounds per square inch (psi).

Upon completion of drilling, the holes were backfilled using cement grout, as required by the SCVWD.

3.2 Cone Penetration Tests

The CPTs, designated CPT-1 through CPT-3, were performed on 6 March 2008 at the locations shown on Figure 2. The CPTs were advanced to depths of about 90 to 100 feet below existing grades.

The CPTs were performed by Brittsan CPT, Inc. using truck-mounted equipment. The CPTs were performed by hydraulically pushing a 1.4-inch-diameter (10 square centimeters), cone-tipped probe into the ground. The cone on the end of the probe is equipped to measure tip resistance, and the sleeve behind the cone tip measures frictional resistance. Electrical strain gauges within the cone measure soil parameters continuously for the entire depth advanced. Penetration data were transferred to a computer and processed to provide engineering information, such as the type of soil encountered and its approximate strength characteristics. Downhole shear wave velocity readings were taken at five-foot depth intervals in CPT-1 and CPT-3.

The CPT logs, which show tip resistance and friction ratio with depth, as well as interpreted SPT N-value, undrained shear strength, friction angle, and soil type, are presented as Figures B-1 through B-3 in Appendix B. The results of the shear wave velocity measurements are shown on Figures B-4 and B-5. Soil encountered by the CPTs was classified in accordance with the chart presented on Figure B-6.

4.0 LABORATORY TESTING

We re-examined the soil samples obtained from our borings to confirm field classification and select representative samples for testing. Our laboratory testing program was designed to correlate soil properties and to evaluate engineering properties of the soil at the site. Samples were tested to measure moisture content, percent fines (material passing the No. 200 sieve), shear strength, and compressibility. The results are summarized on the boring logs and on Figures C-1 through C-3 (Appendix C).

5.0 SUBSURFACE CONDITIONS

The results of our investigation indicate the site is underlain by alluvial deposits consisting predominantly of clay and sandy clay layers interbedded with sand and clayey sand layers to the maximum explored depth of about 100 feet. The near-surface clay (upper approximately five feet) is very stiff and moderately to highly expansive[2]. The clay layers below a depth of five feet are generally medium stiff to stiff to a depth of about 40 feet, where they become very stiff. Discontinuous and localized medium dense to dense sand and gravel layers with varying fines contents were encountered at depths from about 10 to 100 feet across the site. The thickness of the medium dense to dense sand and gravel layers varies from about 2 inches to 7 feet.

Groundwater was encountered at a depth of approximately 13 feet in Boring B-1, corresponding to Elevation 29 feet. The groundwater level was measured in the boring after bailing the drilling fluid from the hole and allowing it to remain open overnight. Seasonal fluctuations influence groundwater levels and may cause several feet of variation in the groundwater level. On the basis of current groundwater measurements, we preliminarily judge a design groundwater level of Elevation 31 feet is appropriate for this site.

6.0 SEISMIC CONSIDERATIONS

6.1 Regional Seismicity

The major active faults in the area are the San Andreas, San Gregorio, Hayward, and Calaveras Faults. These and other faults of the region are shown on Figure 3. For each of the active faults within about 50 kilometers of the site, the distance from the site and estimated mean characteristic Moment magnitude[3] [2007 Working Group on California Earthquake Probabilities (WGCEP) (2007) and Cao et al. (2003)] are summarized in Table 1.

2 Highly expansive soil undergoes large volume changes with changes in moisture content.

3 Moment magnitude is an energy-based scale and provides a physically meaningful measure of the size of a faulting event. Moment magnitude is directly related to average slip and fault rupture area.

TABLE 1
Regional Faults and Seismicity

Fault Segment	Approximate Distance from Site (km)	Direction from Site	Mean Characteristic Moment Magnitude
Monte Vista-Shannon	11	Southwest	6.8
Hayward - South East Extension	12	Northeast	6.4
South Hayward	14	Northeast	6.7
Total Hayward	14	Northeast	6.9
Total Hayward-Rodgers Creek	14	Northeast	7.3
Total Calaveras	16	East	6.9
San Andreas - 1906 Rupture	17	Southwest	7.9
San Andreas – Peninsula	17	Southwest	7.2
San Andreas - Santa Cruz Mnts.	22	South	7.0
Sargent	26	South	6.8
Zayante-Vergeles	32	South	6.8
Northern San Gregorio	38	West	7.2
Total San Gregorio	38	West	7.4
Greenville	40	East	6.9
Mt Diablo – MTD	42	Northeast	6.7

Figure 3 also shows the earthquake epicenters for events with magnitude greater than 5.0 from January 1800 through December 2000. Since 1800, four major earthquakes have been recorded on the San Andreas Fault. In 1836 an earthquake with an estimated maximum intensity of VII on the Modified Mercalli (MM) scale (Figure 4) occurred east of Monterey Bay on the San Andreas Fault (Toppozada and Borchardt 1998). The estimated Moment magnitude, $M_{w,}$ for this earthquake is about 6.25. In 1838, an earthquake occurred with an estimated intensity of about VIII-IX (MM), corresponding to a M_w of about 7.5. The San Francisco Earthquake of 1906 caused the most significant damage in the history of the Bay Area in terms of loss of lives and property damage. This earthquake created a surface rupture along the San Andreas Fault from Shelter Cove to San Juan Bautista approximately 470 kilometers in length. It had a maximum intensity of XI (MM), a M_w of about 7.9, and was felt 560 kilometers away in Oregon, Nevada, and Los Angeles. The most recent earthquake to affect the Bay Area was the Loma Prieta Earthquake of 17 October 1989, in the Santa Cruz Mountains with a M_w of 6.9, approximately 38 km from the site.

In 1868 an earthquake with an estimated maximum intensity of X on the MM scale occurred on the southern segment (between San Leandro and Fremont) of the Hayward Fault. The estimated M_w for the earthquake is 7.0. In 1861, an earthquake of unknown magnitude (probably a M_w of about 6.5) was reported on the Calaveras Fault. The most recent significant earthquake on this fault was the 1984 Morgan Hill earthquake (M_w = 6.2).

The 2007 WGCEP at the U.S. Geologic Survey (USGS) predicted a 63 percent chance of a magnitude 6.7 or greater earthquake occurring in the San Francisco Bay Area in 30 years. More specific estimates of the probabilities for different faults in the Bay Area are presented in Table 2.

TABLE 2
WGCEP (2007) Estimates of 30-Year Probability of a Magnitude 6.7 or Greater Earthquake

Fault	Probability (percent)
Hayward-Rodgers Creek	31
N. San Andreas	21
Calaveras	7
San Gregorio	6
Concord-Green Valley	3
Greenville	3
Mount Diablo Thrust	1

6.2 Geologic Hazards

During a major earthquake on a segment of one of the nearby faults, strong to very strong shaking is expected to occur at the site. Strong shaking during an earthquake can result in ground failure such as that associated with soil liquefaction[4], lateral spreading[5] and differential compaction[6]. Each of these hazards is discussed in the following subsections.

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4 Liquefaction is a phenomenon in which saturated, cohesionless soil experiences a temporary loss of strength due to the buildup of excess pore water pressure, especially during cyclic loading such as that induced by earthquakes. Soil most susceptible to liquefaction is loose, clean, saturated, uniformly graded, fine-grained sand and silt of low plasticity that is relatively free of clay.

5 Lateral spreading is a phenomenon in which surficial soil displaces along a shear zone that has formed within an underlying liquefied layer. Upon reaching mobilization, the surficial blocks are transported downslope or in the direction of a free face by earthquake and gravitational forces.

6 Differential compaction is a phenomenon in which non-saturated, cohesionless soil is compacted by earthquake vibrations, causing differential settlement.

6.2.1 Liquefaction and Lateral Spreading

Saturated, cohesionless soil can liquefy as it experiences a temporary loss of shear strength created by a transient rise in excess pore pressure generated by strong ground motion. Flow failure, lateral spreading, differential settlement, loss of bearing strength, ground fissures, lurch cracking, and sand boils are evidence of liquefaction. The State of California Seismic Hazard Zones maps, San Jose West Quadrangle, Official Map dated 7 February 2002, indicates the site is within a potentially liquefiable area.

Layers of loose to medium dense saturated silty sand and medium stiff sandy silt, varying in thickness from approximately 2 inches to 7 feet, were encountered below the historic high groundwater table to approximately 80 feet below existing ground surface. Based on our analyses, we conclude several of these layers could potentially liquefy during a major earthquake and may experience liquefaction-induced settlement.

Using the Tokimatsu and Seed (1987) method for evaluating earthquake-induced liquefaction settlement, we estimate settlement on the order of 1/4 to 1-1/2 inches may occur beneath the site. Typically, differential settlements are on the order of 1/2 of the total settlements. The results of our liquefaction and seismic settlement studies are shown in Table 3.

TABLE 3
Estimated Liquefaction-Induced Settlement

CPT/Boring Location	Approximate Intervals of Liquefiable Soil (depth, feet bgs)	Total Settlement (inches)
CPT-1	10.8 – 11.3 12.1 – 12.3 38.4 – 38.6 52.8 – 53.0 74.3 – 74.5 77.9 – 78.2	1/4
CPT-2	39.7 – 39.9 42.1 – 42.2 44.5 – 44.6 45.6 – 45.7 62.9 – 63.0 76.2 – 76.3	1/4
CPT-3	17.4 – 17.7 19.6 – 19.7 21.2 – 21.3 26.2 – 26.7 37.5 – 37.7 38.5 – 38.8 39.2 – 39.9 40.3 – 40.8	1/2
B-1	39 - 41	1/4
B-2	19 – 22 37 - 43	1-1/4
B-3	10 – 13.5 39 - 46	1-1/2

Because of the relatively thin and discontinuous nature of the layers that could potentially generate excess pore pressure, the seismic ground response at the site should not be significantly affected and the site should behave like a deep stiff soil site. In addition, the driving of piles to support the new office structures should densify the sand layers beneath these buildings, thereby reducing the potential for generation of excess pore pressure in the vicinity of the office structures during a major earthquake. Furthermore, because of the relatively discontinuous nature of the layers that could potentially generate excess pore pressure, our preliminary conclusion is that the potential for lateral spreading at the site is low.

6.2.2 Differential Compaction

Seismically-induced compaction or densification of non-saturated sand (sand above the groundwater table) due to earthquake vibrations can result in settlement of the ground surface. Considering the soil above the groundwater table is predominately clay or the sand layers have high clay content and/or are sufficiently dense, our preliminary conclusion is the potential for ground settlement due to differential compaction is low.

6.2.3 Ground Rupture

Historically, ground surface displacements closely follow the trace of geologically young faults. The site is not within an Earthquake Fault Zone, as defined by the Alquist-Priolo Earthquake Fault Zoning Act, and no known active or potentially active faults exist on the site. In a seismically active area, a remote possibility exists for future faulting in areas where no faults previously existed; however, we conclude the risk of surface faulting and consequent secondary ground failure is low.

7.0 DISCUSSION AND PRELIMINARY CONCLUSIONS

The primary geotechnical issues to be addressed for the project are potential settlement, including consolidation settlement of clay under static building loads and potential liquefaction-induced settlement during a major earthquake, selecting the most appropriate foundation system for the proposed structures, and the presence of groundwater within the proposed excavation depths of the parking structures. Our preliminary conclusions regarding these and other issues are discussed in the remainder of this section.

7.1 Foundation and Settlement

Selecting the most appropriate foundation type for the proposed new office and parking garage structures depends on the anticipated structural loads, amount of differential settlement the structures can tolerate, the bearing capacity of the foundation soil, construction costs, and access restrictions.

Plans indicate the proposed garage structures on both parcels will have two below-grade levels, while the two proposed 7-story office structures (San Tomas parcel) and the 8-story office structure (Condensa parcel) will be constructed at-grade (no basements). Our discussion regarding foundations and associated estimated settlement for each proposed structure is presented in the remainder of this section.

7.1.1 Parking Garage Structures

Because the proposed excavation for the two below-grade levels for the garage structures will result in a net decrease in overburden pressure under the parking garage footprint, we do not anticipate excessive settlements due to consolidation of underlying clay layers. Furthermore, the basements will extend below the groundwater level and the foundation system/floor will need to resist hydrostatic uplift pressures and span between columns. Considering these issues, we conclude the parking garage structures should be supported on mat foundations. Tiedowns should be provided to resist the hydrostatic uplift pressures if the weight of the mat and building are insufficient or the mat cannot adequately span between columns. A preliminary design groundwater level at Elevation 31 feet should be used to estimate hydrostatic pressures.

We anticipate the garage structures, however, will settle moderately due to recompression of the soil under the building loads; we estimate total static settlement could be on the order of one inch. The majority of the anticipated settlement should occur during construction. As discussed in Section 6.2.1, an additional 1/4 to 1-1/2 inch of seismically-induced total settlement should also be expected with a differential settlement of about 1/2 inch between columns. These estimated settlements are based on widely spaced borings and CPTs; additional field exploration performed during a detailed final geotechnical investigation should provide more precise estimates of total and differential settlements. Furthermore, the estimated differential settlements do not take into account the rigidity of the mat, and therefore the actual differential settlement should be less.

Because the basement walls and mat will extend below the groundwater level, they should be waterproofed.

7.1.2 Office Buildings

If the proposed office buildings were supported on a shallow foundation, we conclude the settlement due to consolidation of the clay below would be excessive due to the relatively high building loads. Therefore, we conclude the proposed office buildings should be supported on deep foundations, consisting of driven piles gaining support in friction along the shaft. Typically, 14-inch square, prestressed, precast, concrete piles are the most economical piles in the Bay Area. The settlement of properly constructed driven piles, designed based on the preliminary recommendations presented herein, should be less than 1/2 inch. Differential settlement between adjacent pile caps should be less than ¼ inch. As discussed in Section 6.2.1, an additional 1/4 to 1-1/2 inches of seismically-induced total settlement should also be expected with a differential settlement of about 1/2 inch between columns. Because the building is pile-supported, it should not affect the structure; however, unless the ground floor slab is a structural slab, designed to span between the pile caps, a slab-on-grade could settle and crack. If this is objectionable, a structural slab may be required.

7.2 Dewatering, Excavation, Shoring, and Underpinning

Our preliminary conclusions regarding dewatering, shoring, and excavation are discussed in the following subsections.

7.2.1 Dewatering

To construct the proposed basement levels for the parking structures, the groundwater should be lowered to a depth of at least three feet below the bottom of the planned excavation. A site dewatering system should be designed, installed and operated by an experienced dewatering contractor. However, we should review the dewatering system proposed by the contractor prior to installation. Special care should be taken to reduce the removal of fines from the granular layers. The dewatering should be maintained until sufficient building weight and/or tiedown capacity is available to resist the hydrostatic uplift forces on the bottom of the foundation, as directed by the project structural engineer.

Variables that influence the performance of the dewatering system and the quantity of water produced include the number of wells, the depth and positioning of the wells, the interval over which each well is screened, and the rate at which each well is pumped. Different combinations of these variables can be used to successfully dewater the site. Because of the size of each garage excavation and the presence of clayey soil, a system of perimeter wells may not sufficiently dewater the excavation. Interior wells may also be needed to adequately dewater the site and minimize disturbance to the subgrade.

A working pad of gravel, as discussed in Section 7.3 and 8.1.1, can also be used as a temporary drainage blanket in addition to wells. Perforated pipes may be placed in the gravel to collect water and conduct it to a sump or other appropriate outlet.

Dewatering for the parking structure excavations should remain as localized as possible. Widespread dewatering could result in subsidence of the area around the excavations due to increases in effective stress in the soil. Nearby streets and other improvements should be monitored for vertical movement and groundwater levels outside the excavation should be monitored through wells while dewatering is in progress. Should excessive settlement or groundwater drawdown be measured, the contractor should be prepared to recharge the groundwater outside the excavation through recharge wells. A recharge program should be submitted as part of the dewatering plan.

7.2.2 Excavation and Shoring

We anticipate construction of two levels of below-grade parking will require an excavation of about 20 to 25 feet below existing grades. The shoring design should also take into account the overexcavation of 12 inches across the site to create a gravel working pad, as discussed in Section 7.2.1, for the mat foundation. We anticipate the soil to be excavated from the site can be excavated with conventional earthmoving equipment, such as loaders and backhoes.

Where sloping of the excavation is not feasible, considering the height of the cut and the expected soil conditions, we conclude that a soldier-pile-and-lagging shoring system is the most suitable for this project. We considered soil nailing, but do not recommend this system for this site for the following reasons:

- The excavation will extend about 10 feet below the design groundwater level
- Dewatering is generally not effective in reducing the pore water pressure in clays
- There is a tendency in stiff, high plasticity clays, which are present across a significant portion of the site, for tension cracks to be present; the presence of tension cracks significantly diminishes the shear strength of the soil, especially below the groundwater table
- We anticipate sand/gravel layers that could be susceptible to caving will be present within the depths of the proposed excavations

A soldier-pile-and-lagging system consists of steel soldier beams, placed in vertical predrilled holes that are backfilled with concrete and wood lagging installed between the soldier beams as the excavation proceeds. Lateral resistance against movement may be mobilized by extending the soldier beams below the bottom of the excavation. Typically, to restrict potential wall movement, tiebacks are installed for an excavation depth greater than 10 to 15 feet. If tiebacks are used to provide lateral support for the shoring, care should be taken to locate utilities and other possible underground obstructions prior to installation. Additionally, permits and/or the permission of the owner will likely be required to encroach on neighboring property and streets.

The selection, design construction, and performance of the shoring system should be the responsibility of the contractor and the shoring designer, who can be retained by either you or the contractor.

During excavation, the shoring system is expected to yield and deform, which could cause surrounding improvements to settle. The magnitude of shoring movements and resulting settlements are difficult to estimate because they depend on many factors, including the method of installation and the contractor's skill in the shoring installation. We believe that movements of a properly designed and constructed shoring system should be within ordinary accepted limits of one inch at the top of the wall. Some deformation of the ground surface could also occur if the sides of the excavation are sloped. A monitoring program should be established to evaluate the effects of the construction on any adjacent underground utilities, paved surfaces, and adjacent improvements.

7.3 Mat Subgrade

Because the bottom of the excavation for the garage structures will be below the groundwater level, the soil at subgrade level will be near saturation even after dewatering. Additionally, the clay exposed at the foundation and slab level will be susceptible to disturbance under construction equipment loads. To help protect the soil subgrade, a working pad should be constructed. The pad should consist of open graded crushed rock and should be underlain by a geotextile fabric. This layer of crushed rock can also be used as part of the dewatering system. Waterproofing should be placed as discussed in Section 8.1.1 or per the manufacturer's specifications.

7.4 Construction Considerations

We encountered moderately to highly expansive near-surface soil in some areas of the site during our investigation. Where concrete slabs-on-grade will be constructed, expansive soil should be removed or special measures taken to mitigate its detrimental effects. This can be accomplished by moisture conditioning the expansive soil beneath all slabs-on-grade to above the optimum moisture content, placing a non-expansive select fill layer on which to support the slabs, and sufficiently reinforcing the slabs. The near-surface soil most likely does not meet the criteria for non-expansive select fill.

In addition, the near-surface clay may be susceptible to pumping and rutting during construction, especially if it becomes wet. If localized soft or wet areas are encountered, it may be necessary to overexcavate them to a depth of 18 to 24 inches, place a geotextile fabric, such as Mirafi 500X or equivalent, at the bottom of the overexcavation and backfill with granular material to stabilize the subgrade and bridge the soft material. If grading of the site is performed during the rainy season, typically between November and April, lime treatment or other soil stabilization techniques may be required to provide a stable, workable subgrade for grading operations or pile driving equipment, including forklifts.

8.0 PRELIMINARY RECOMMENDATIONS

Preliminary recommendations for site preparation, shoring design, foundation support, below-grade walls, tiedown anchor design, seismic design, and other issues are presented in the following sections of this report. The recommendations provided herein are based on limited subsurface investigation, are intended to be preliminary, and should not be used for final design; final recommendations will be provided following a detailed geotechnical investigation at the site.

8.1 Site Preparation

Existing pavements, buildings, old foundations, abandoned utilities, and other obstructions should be removed from areas to receive improvements. We anticipate the excavation for this project can be made using conventional earth-moving equipment. Where utilities to be removed extend off site, they should be capped or plugged with grout at the property line. It may be feasible to abandon utilities in-place outside the proposed building footprint, provided they will not interfere with future utilities or building foundations. If utilities are abandoned in-place, they should be capped at the ends and completely filled with flowable cement grout over their entire length. Existing utility lines, where encountered, should be addressed on a case-by-case basis.

8.1.1 Mat Foundation Subgrade Preparation

Because the excavation for the parking garage structures will extend below the groundwater level, the soil at subgrade level will be near saturation even after dewatering. To protect the subgrade, we recommend heavy construction equipment not be allowed within 3 feet of subgrade elevation and that final excavation be made with excavators or backhoes with smooth buckets. Without an extended period for drying, we judge the subgrade may not support even light equipment and foot traffic without experiencing excessive disturbance.

To help protect the subgrade we recommend overexcavating the site and creating a gravel working pad on which to construct the mat. We anticipate an overexcavation of 12 inches will suffice if used in conjunction with a woven reinforcing fabric (geotextile), such as Mirafi 500X. After placing the reinforcing fabric on the exposed subgrade, the overexcavation should be backfilled with clean 3/4- to 1-inch gradation crushed rock.

Because the proposed foundation will be below the groundwater level, waterproofing the base of the mat is recommended. We recommend the waterproofing be placed either directly on the crushed gravel or on a mud slab (thin layer of lean concrete) and be covered by a mud slab. The mud slabs, which should be at least three inches thick, should reduce the potential for damage to the waterproofing and provide a firm, smooth surface on which to place the reinforcing steel for the mat and structural slab. We recommend the waterproofing be placed in accordance with the manufacturer's specifications. If they differ from our recommendations, the manufacturer's specification should be followed to preserve their warranty.

As discussed in Section 7.3, depending on the amount of water at the subgrade elevation, it may be desirable to use the crushed rock working pad as a temporary drainage blanket. To drain the crushed rock, four-inch diameter perforated PVC pipe should be placed near the bottom of the gravel, spaced every 30 feet, to direct water trapped in the gravel to a sump. The sump should be properly abandoned before the completion of construction.

The soil subgrade at the base of the excavation should be free of standing water, debris, and disturbed materials prior to placing the reinforcing fabric and crushed rock. If loose material is observed in the excavation, it should be overexcavated to firm, competent material and replaced with crushed rock or lean concrete. We should check the exposed subgrade after cleaning, but prior to placement of the working pad, mud slab, or waterproofing.

8.1.2 At-Grade Improvements

We anticipate moderately to highly expansive soil to be present near existing grades. At-grade areas that will receive improvements (including building floor slabs, sidewalks, and exterior concrete flatwork) should be stripped of existing improvements. In areas to receive improvements, the surface exposed by stripping or excavation should be scarified to a depth of at least twelve inches, moisture-conditioned to at least three percent above optimum moisture content, and compacted to between 88 and 93 percent relative compaction[7] to reduce its expansion potential. The scarification, moisture conditioning, and compaction should extend at least two feet beyond the slabs. Where slabs-on-grade are used, we recommend at least 12 and 18 inches of imported (select) material, as described below, be placed where expansive soil is exposed beneath exterior and interior slabs, respectively; the select fill should also extend at least two feet beyond the slab edges. If soft or loose soil is encountered, the unsuitable material should be removed and be replaced with suitable fill material that is properly compacted and moisture conditioned. The exposed ground surface should be kept moist during subgrade preparation.

Fill should be non-corrosive, non-hazardous, free of organic matter, contain no rocks or lumps larger than three inches in greatest dimension, have a liquid limit less than 40 and plasticity index less than 12, have at least 20 percent fines content (minus #200 sieve), and be approved by the geotechnical engineer. All fill should be placed in lifts not exceeding eight inches in loose thickness, moisture-conditioned to near optimum moisture content, and compacted to at least 90 percent relative compaction. The subgrade surface should be rolled to a dense, non-yielding surface. If the compacted subgrade is disturbed during utility trench or foundation excavations, the subgrade should be re-rolled to provide a smooth, firm surface for concrete slab support.

7 Relative compaction refers to the in-place dry density of soil expressed as a percentage of the maximum dry density of the same material, as determined by the ASTM D1557 laboratory compaction procedure.

8.2 Excavation, Temporary Slopes, and Shoring

We anticipate that construction of the proposed parking garage below-grade levels will require an excavation of about 20 to 25 feet. Where space permits and the slope is sufficiently dewatered, the excavation may be open cut with sloping sides. Temporary slopes should not be steeper than 1-1/2:1 (horizontal to vertical) in soil. The slope should be dewatered to prevent seepage along the face of the excavation and maintain stability. All slopes should be monitored during excavation to verify their stability.

Where space restrictions preclude the use of temporary cut slopes, the shoring system, consisting of cantilevered or tied-back soldier-pile-and-lagging shoring systems may be installed. Our recommendations for this shoring system are presented in the following sections.

8.2.1 Soldier-Pile-and-Lagging

Cantilevered soldier pile and lagging shoring should be designed to resist active pressures calculated using an equivalent fluid weight of 35 pcf (assumes level ground surface being retained). The active pressure should be extended to the bottom of the soldier beams. Tied-back soldier piles and lagging shoring should be designed to resist the pressures presented on Figure 5. These earth pressures are based on fully dewatered conditions. Traffic or surcharge loads should be added to the active pressures. If traffic loads are expected within 10 feet of the walls, an additional design load of 100 psf should be applied to the upper 10 feet of the walls.

Passive resistance below the bottom of the excavation may be computed using a uniform pressure of 1,500 pounds per square foot (psf). Passive pressures can be assumed to act on an area of three pile widths, provided the soldier piles are spaced at least three diameters apart. This value includes a factor of safety of 1.5 and assumes the groundwater level will be at least 3 feet below the bottom of the excavation.

The shoring designer should evaluate the required penetration depth of the soldier piles. The soldier piles should have sufficient axial capacity to support the vertical load acting on the piles, if any. To compute the axial capacity of the piles, we recommend using an allowable friction of 700 psf in the soil on the perimeter of the piles below the excavation level.

8.2.2 Tiebacks

Tiebacks should derive their load-carrying capacity from the soil and behind an imaginary line sloping upward from a point H/5 feet away from the bottom of the excavation at an angle 60 degrees from horizontal, where H is the wall height in feet, as shown on Figure 5.

Allowable capacities of the tiebacks will depend on the installation method, hole diameter, grout pressure, and workmanship. For estimating purposes, we recommend using a preliminary skin friction value of 700 psf for gravity placed grouted tiebacks or 1,200 psf for pressure-grouted tiebacks within the bond length, with a minimum bond length of 15 feet. The stressing (unbonded) length should be at least 10 and 15 feet for steel bar and strands/tendons, respectively. These values include a safety factor of approximately 1.5. To prevent caving, a Klemm-type rig (double cased hole) should be used to drill the shafts and the tiebacks should be equipped with post-grout tubes.

Determining the length of tieback required to resist the earth pressures presented above should be the contractor's responsibility. The computed bond length should be confirmed by a testing program under our observation. Testing procedures should follow those described in Section 8.2.3 for tieback testing.

If any tiebacks fail to meet the testing requirements, additional tiebacks should be added to compensate for the deficiency as required by the shoring designer. Additionally, the tiebacks should be checked 24 hours after initial prestressing to check that stress relaxation has not occurred. The bottom of the excavation should not extend more than two feet below a row of unsecured tiebacks.

8.2.3 Tieback Testing

We should observe tieback testing. The first two production tiebacks and two percent of the remaining tiebacks should be performance-tested to at least 1.25 times the design load. The remaining tiebacks should be confirmed by proof tests also to at least 1.25 times the design load.

8.2.3.1 Performance Test

The movement of each tieback should be monitored with a free-standing, tripod-mounted dial gauge during performance and proof testing. The performance test is used to verify the capacity and the load-deformation behavior of the tiebacks. It is also used to separate and identify the causes of tieback movement, and to check that the designed unbonded length has been established. In the performance test, the load is applied to the tieback in several cycles of incremental loading and unloading. During the test, the tieback load and movement are measured. The maximum test load should be held for a minimum of 10 minutes, with readings taken at 0, 1, 3, 6, and 10 minutes. If the difference between the 1- and 10-minute reading is less than 0.04 inch during the loading, the test is discontinued. If the difference is more than 0.04 inch, the holding period is extended by 50 minutes to 60 minutes, and the movements should be recorded at 15, 20, 25, 30, 45, and 60 minutes.

8.2.3.2 Proof Test

A proof test is a simple test used to measure the total movement of the tieback during one cycle of incremental loading. The maximum test load should be held for a minimum of 10 minutes, with readings taken at 0, 1, 2, 3, 6, and 10 minutes. If the difference between the 1- and 10-minute reading is less than 0.04 inch, the test is discontinued. If the difference is more than 0.04 inch, the holding period is extended by 50 minutes to 60 minutes, and the movements should be recorded at 15, 20, 25, 30, 45, and 60 minutes.

8.2.3.3 Acceptance Criteria

We should evaluate the tieback test results and, in association with the shoring designer, evaluate tieback acceptability. A performance- or proof-tested tieback with a ten-minute hold is acceptable if the tieback carries the maximum test load with less than 0.04 inch movement between one and 10 minutes, and total movement at the maximum test load exceeds 80 percent of the theoretical elastic elongation of the unbonded length.

A performance- or proof-tested tieback with a 60-minute hold is acceptable if the tieback carries the maximum test load with less than 0.08 inch movement between six and 60 minutes, and total movement at the maximum test load exceeds 80 percent of the theoretical elastic elongation of the unbonded length.

Tiebacks that failed to meet the first criterion may be assigned a reduced capacity. If the total movement of the tiebacks at the maximum test load does not exceed 80 percent of the theoretical elastic elongation of the unbonded length, the contractor may be required to replace the tiebacks.

8.3 Foundation Support

We conclude the proposed parking garage structures may be supported on mat foundations bottomed on stiff clay. The proposed 7- and 8-story office structures should be supported on a deep foundation system consisting of driven precast, prestressed, 14-inch square, concrete piles that gain support from soil friction along the sides of the pile. Details and preliminary recommendations for each foundation system are provided in the following subsections.

8.3.1 Mat Foundation (Parking Garages)

As discussed in Section 7.1, we estimate total static settlement under the anticipated parking garage loads will be less than one inch. Differential settlement will depend on the rigidity of the mat. To design the mat using the modulus of subgrade reaction method, we recommend a modulus of subgrade reaction of 45 kips per cubic foot (kcf). The modulus value is representative of the anticipated settlement under the building loads. After the mat analysis is completed, we should review the computed settlement and bearing pressure profiles to check that the modulus value is appropriate. The modulus is applicable for allowable dead plus live loads up to 3.5 kips per square foot (ksf), and for total loads including seismic of 4.7 ksf. In addition to the static settlement, the mat should be designed for an additional ½-inch of differential settlement between columns during a major earthquake.

Resistance to lateral loads can be mobilized by a combination of passive pressure acting against the vertical faces of the mat and friction along the base of the mat. Passive resistance may be calculated using lateral pressures corresponding to a uniform pressure of 1,500 pounds per square foot (psf). Frictional resistance should be computed using a base friction coefficient of 0.2; this friction value assumes a waterproofing membrane is placed below the mat. These values include a factor of safety of about 1.5 and may be used in combination without reduction.

We recommend a design groundwater level corresponding to Elevation 31 feet be used to evaluate hydrostatic uplift pressure. Tiedowns should be provided where the weight of the building is insufficient to overcome the uplift. Because the mat will be below the design groundwater level, we recommend that it be waterproofed. Our recommendations regarding waterproofing are provided in Section 8.1.1; however, a waterproofing consultant should be retained to provide recommendations for the type of waterproofing and its installation.

The exposed subgrade for the basement should be free of standing water, debris, and disturbed materials prior to constructing a working pad. We should check the mat subgrade after cleaning, but prior to placement of waterproofing, mud slab, crushed rock, or reinforcing steel to confirm bearing and that loose to disturbed material has been removed. If loose or disturbed material is observed in the mat excavation, it should be overexcavated to firm, competent material and be replaced with lean concrete.

8.3.2 Driven Precast, Prestressed, Concrete Piles (Office Buildings)

The proposed office buildings may be supported on driven precast, prestressed, 14-inch square, concrete piles that gain support from friction between the sides of the pile and the soil. Preliminary recommendations for axial and lateral pile capacities, as well as an indicator program, are presented in the following subsections.

8.3.2.1 Axial Load Capacity

Pile lengths should be estimated using the length versus axial capacity curve shown on Figure 6, and confirmed by an indicator pile program. For short term compressive axial loading conditions such as wind or seismic, the capacities shown on Figure 6 may be increased by 1/3. The seismic uplift capacity should be considered to be equal to the allowable compressive axial capacity. To avoid capacity reduction due to group effects, piles should be spaced no closer than three pile widths, center to center.

8.3.2.2 Lateral Load Resistance

The piles should develop lateral resistance from the passive pressure acting on the upper portion of the piles and their structural rigidity. The allowable lateral capacity of the piles depends on:

- the pile stiffness
- the strength of the surrounding soil
- axial load on the pile
- the allowable deflection at the pile top and the ground surface
- the allowable moment capacity of the pile.

We developed load versus deflection and load versus moment curves based on 0.5 and 1 inch of lateral deflection at the top of the pile for both fixed- and free-head conditions for 14-inch square, precast, prestressed concrete piles. These curves are presented on Figures 7 through 10. These lateral capacities are for single piles only. To account for group effects, the lateral load capacity of a single pile should be multiplied by the appropriate reduction factors shown on Table 4. The reduction factors are based on a minimum pile spacing of three widths, center to center.

TABLE 4
Lateral Group Reduction Factors

Number of Piles within Pile Cap	Lateral Group Reduction Factor
2 – 3	0.85
4 – 5	0.8
≥ 6	0.7

The moment profile for a single pile with an unfactored load should be used to check the design of individual piles in a group.

Additional lateral load resistance can be developed by passive resistance acting against the faces of the pile caps and grade beams. Passive resistance may be calculated using lateral pressures corresponding to a uniform pressure of 1,500 pounds per square foot (psf); however, the upper foot of resistance should be neglected unless confined by a slab or pavement. This value includes a factor of safety of 1.5.

8.3.2.3 Indicator Pile Program

We recommend an indicator pile program be performed to provide data for choosing production pile lengths for the precast concrete piles. We recommend driving a minimum of 10 indicator piles within each of the proposed office building footprints. Indicator piles may be installed at production pile locations selected by us and approved by the structural engineer. They should be installed with the same equipment that will be used to install the production piles. Indicator piles should be a minimum of 10 feet longer than the estimated design length (as determined using Figure 6). If indicator piles are driven from existing grade, then a follower may be used; the length of the follower will depend on the design cutoff elevation relative to existing grade.

To reduce the amount of spoils and not reduce the axial capacity, predrilling to allow for alignment of the pile should not extend more than about five feet below grade. The auger diameter used for predrilling should be no greater than 14 inches.

We recommend attaching pile driving analyzer (PDA) transducers to at least three indicator piles within each building footprint; the locations of which should be selected by us before driving. The pile integrity and dynamic capacity of these piles should be monitored with the PDA during initial driving and retap. A Case Pile Wave Analysis Program (CAPWAP) should be performed on one representative blow on each of the selected indicator piles during restrike at least 24 hours after initial driving. To allow for monitoring of the PDA, the pile will need to be left several feet above the ground surface or bottom of the pile cap, if excavated.

Our engineer should be on site full-time to observe the indicator pile driving operation on a continuous basis, and will maintain pile driving records showing resistance to penetration versus depth for each pile. We will evaluate the capacity of the indicator piles using available subsurface data, pile driving records, and dynamic testing data. On the basis of our evaluation, we will provide a summary letter report with final recommendations regarding pile lengths and capacity.

Determining the driving equipment for this project should take into account the "matching" of the pile hammer with the pile size and length, and soil conditions. All piles should be driven continuously to their design tip embedments using a hammer that can deliver sufficient energy to the tip of the piles to drive them efficiently without damage. The hammer should have a maximum rated energy of at least 60,000 foot-pounds or greater; however, it should not exceed 90,000 foot-pounds at a maximum stroke.

8.4 Basement Wall Design

We recommend all basement walls be designed to resist lateral pressures imposed by the adjacent soil and vehicles. In addition, because the site is a seismically active area walls should be designed to resist pressures associated with seismic forces. Basement walls should be designed for the equivalent fluid weights and pressures presented in Table 5, where H is the entire height of the wall in feet.

TABLE 5
Lateral Earth Pressures for Basement Wall Design

	Equivalent Fluid Weights for Static Condition		
	Unrestrained Walls	**Restrained Walls**	**Seismic Condition**
Above the water table (Elevation 31 feet)	35 pcf	55 pcf	35 pcf + 15H psf
Below the water table (Elevation 31 feet)	80 pcf	90 pcf	80 pcf + 15H psf

Walls should be designed for the more critical loading condition of static restrained or seismic condition. Where traffic will pass within 10 feet of basement walls, temporary traffic loads should be considered in the design of the walls. Traffic loads may be modeled by a uniform pressure of 100 psf applied in the upper 10 feet of the walls.

If the basement walls are designed to resist lateral forces such as wind or earthquake loading they should be checked using passive pressures. To calculate the passive resistance against the below-grade walls, we recommend using a maximum uniform pressure of 1,500 psf. This value contains a factor of safety of 1.5.

The lateral earth pressures given assume the walls are properly backdrained above the water table to prevent the buildup of hydrostatic pressure. One acceptable method for backdraining the walls is to place a prefabricated drainage panel against the back side of the wall. The drainage panel should extend down to the design groundwater elevation (Elevation 31 feet) to a perforated PVC collector pipe. The pipe should be surrounded on all sides by at least four inches of Caltrans Class 2 permeable material (Caltrans Standard Specifications Section 68-1.025). We should check the manufacturer's specifications regarding the proposed prefabricated drainage panel material to verify that it is appropriate for the intended use. An acceptable alternative is to backdrain the wall with Caltrans Class 2 material at least one foot wide, extending down to the base of the wall. A perforated PVC pipe should be placed at the bottom of the gravel, as described for the first alternative. The pipe in either alternative should be sloped to drain into an appropriate outlet. We should check the manufacturer's specifications for the proposed drainage panel material to verify it is appropriate for its intended use.

To protect against moisture migration, below-grade walls should be waterproofed and water stops placed at all construction joints. The waterproofing should be placed directly against the backside of the walls and according to manufacturer's specifications.

8.5 Tiedown Anchors

Tiedown anchors may be used where the uplift pressure will exceed the anticipated building loads. Tiedown anchors typically consist of relatively small-diameter, drilled, concrete or grout-filled shafts with steel bars or tendons embedded in the concrete or grout. The anchors develop their uplift resistance from friction between the sides of the shaft and the surrounding soil.

The center-to-center spacing of tiedown anchors should be at least four shaft diameters apart or 4 feet, whichever is greater. The ultimate bond strength between the anchor and soil will depend on the installation procedure. The actual bond strength should be estimated by the designer. For planning purposes, however, we recommend using an ultimate skin friction of 1,500 psf for post-grouted tiedowns. Higher values may be obtained depending upon the techniques employed by the contractor and the results of pullout tests. A safety factor of 1.5 and 2.0 should be used for temporary loads (such as seismic) and permanent loads (such as hydrostatic), respectively.

Special attention should be given to waterproofing the connections between the tiedown anchors and the foundation. Because tiedowns will be permanent, we recommend they be double corrosion protected. The tiedowns will be installed below the water table; therefore, the contractor should use an installation method that prevents the holes from caving. If water is present in the shaft, concrete should be placed using a tremie system. High strength bars or strands may be used as tensile reinforcement in the anchors. A minimum stressing length (free length) of 10 and 15 feet should be provided for bar and strand tendons, respectively.

The first two production tiedowns and two percent of the remaining tiedowns should be performance-tested to 1.5 or 2.0 times the design load, depending on whether temporary or permanent uplift governs. All other tiedowns should be proof-tested to either 1.5 or 2.0 times the design load. The anchors should be tested as recommended in Section 8.2.3 for tiebacks, except for test load. After testing, all anchors should be loaded and locked off to a portion of their design load as determined by the structural engineer and indicated on the structural drawings and/or in the specifications.

8.6 Seismic Design

For seismic design in accordance with the provisions of the 2007 California Building Code (CBC), we recommend using site class D with site coefficient values F_a and F_v of 1.0 and 1.5, respectively.

The mapped site class B short (S_s) and one second (S_1) spectral values for Maximum Considered Earthquake (MCE) for the project site are 1.50g and 0.60g, respectively. Using the site class D modification factors F_a and F_v of 1.0 and 1.5, the corresponding S_{MS} and S_{M1} for the project site are 1.50g and 0.90g.

9.0 LIMITATIONS

The conclusions and recommendations presented in this report result from limited subsurface investigation should be considered preliminary and should not be used for final design. Final design recommendations should follow a detailed geotechnical investigation at the site.

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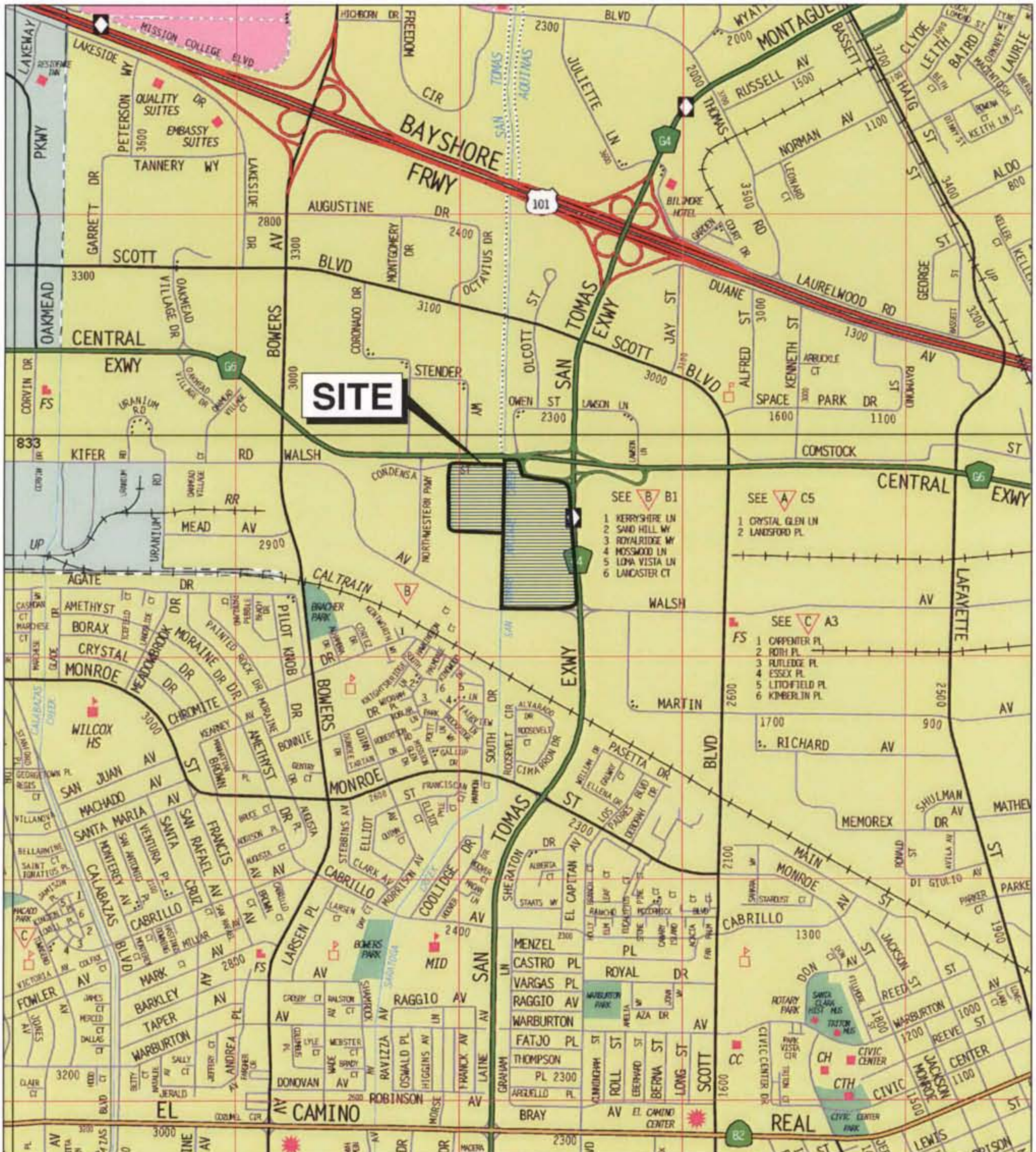
FIGURES



Base map: The Thomas Guide
Santa Clara County
1999

0 1/4 1/2 Mile

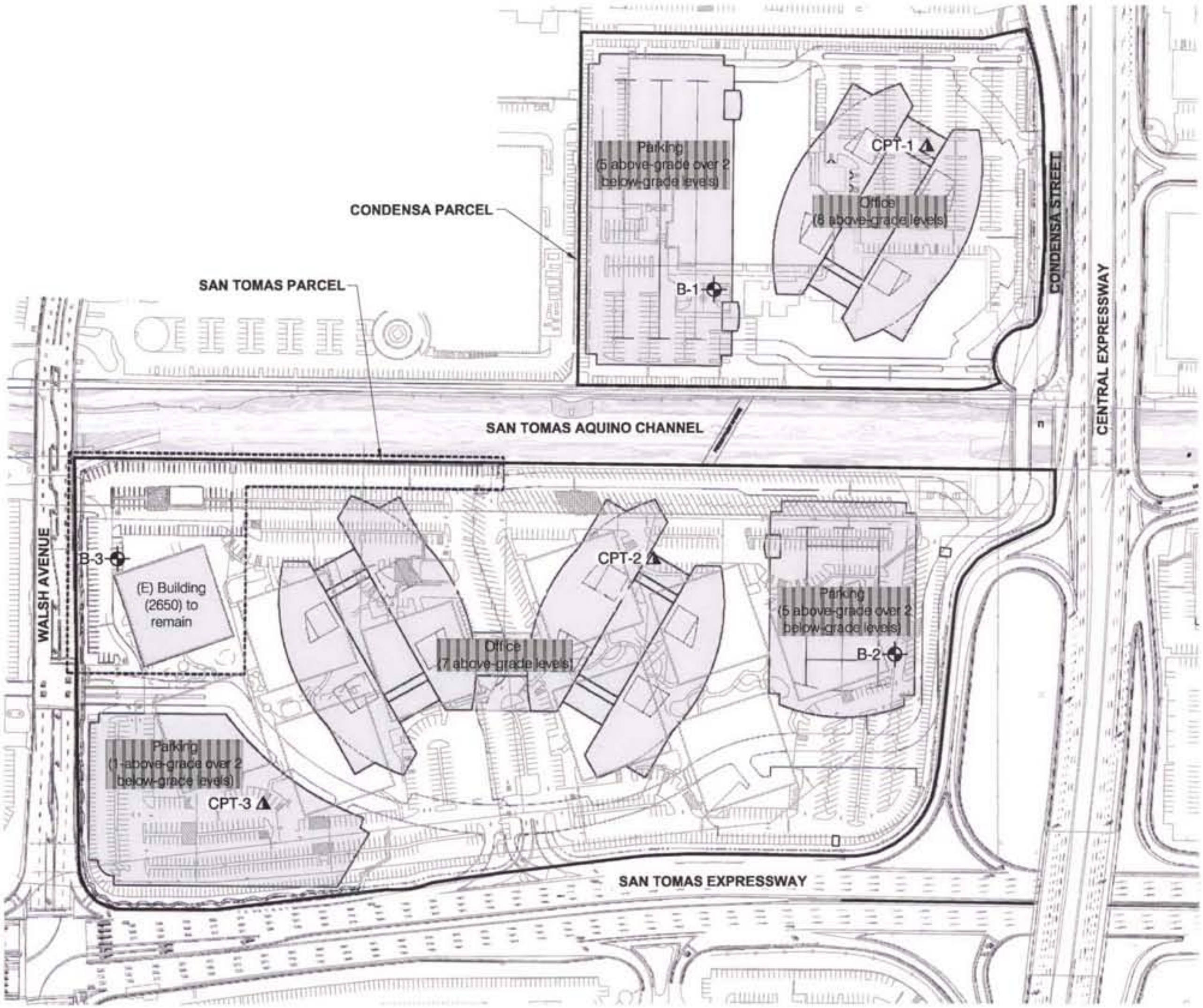
Approximate scale

SAN TOMAS BUSINESS PARK
Santa Clara, California

SITE LOCATION MAP

Treadwell&Rollo

Date 05/20/08	Project No. 4783.01	Figure 1



EXPLANATION

B-1 — Approximate location of boring by Treadwell & Rollo, Inc., March 2008

CPT-1 — Approximate location of cone penetration test by Treadwell & Rollo, Inc., March 2008

0 — 200 Feet

Approximate scale

SAN TOMAS BUSINESS PARK Santa Clara, California		
SITE PLAN SHOWING BORING AND CPT LOCATIONS		
Date 05/20/08	Project No. 4783.01	Figure 2

Treadwell&Rollo

Reference: 1.) Base map from a drawing titled "Topographic Survey, for Harvest Properties", by Kier & Wright, Civil Engineers & Surveyors, Inc., dated January 2008.
2.) San Tomas Business Park Campus, Santa Clara, California, Phasing Option A, Phase 3, by Korth Sunseri Hagey Architects, dated 6 May 2008.

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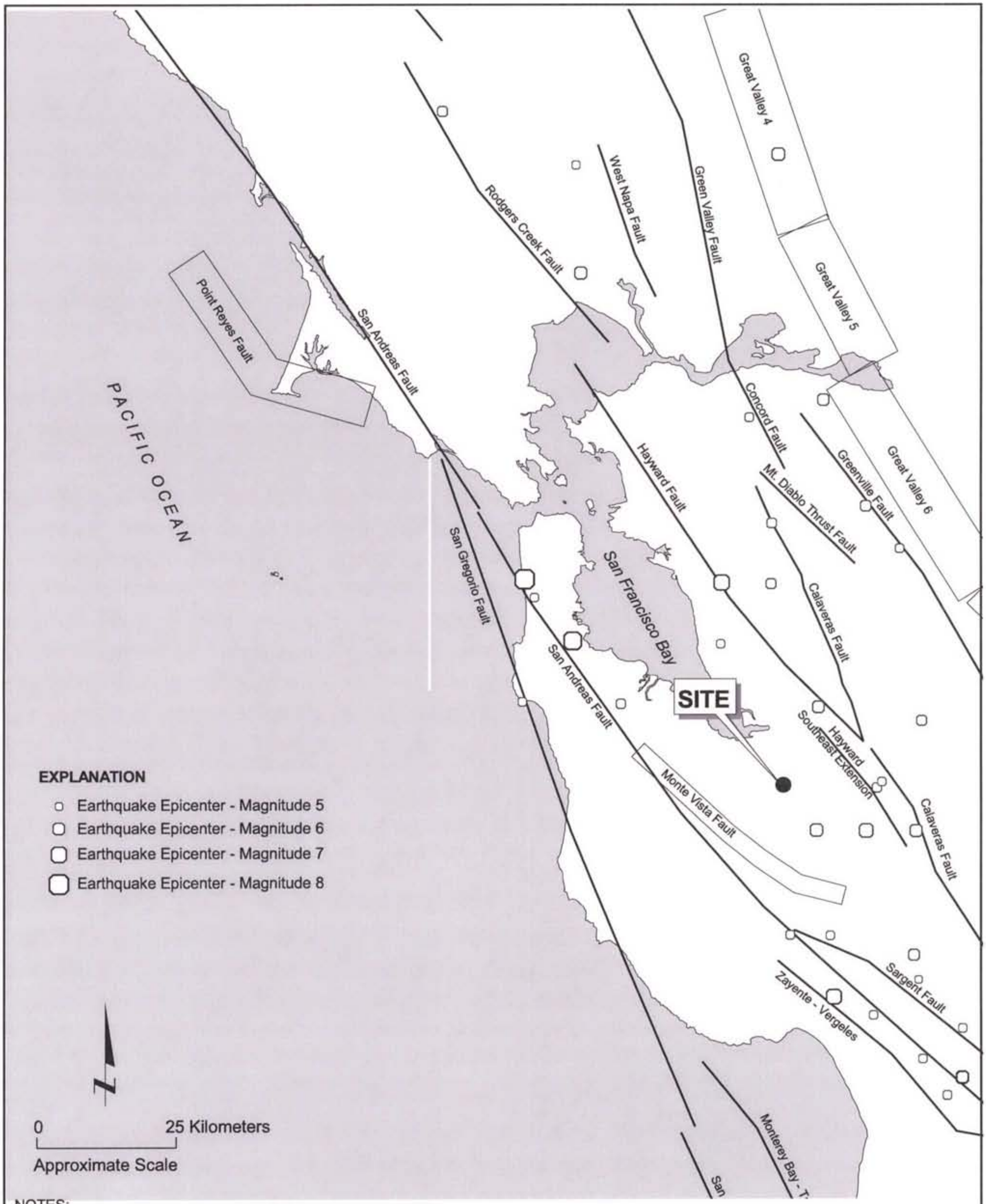


NOTES:
Digitized data for fault coordinates and earthquake catalog was developed by the California Department of Conservation Division of Mines and Geology. The historic earthquake catalog includes events from January 1800 to December 2000.

SAN TOMAS BUSINESS PARK Santa Clara, California	**MAP OF MAJOR FAULTS AND EARTHQUAKE EPICENTERS IN THE SAN FRANCISCO BAY AREA**
Treadwell&Rollo	Date: 05/20/08 | Project No. 4783.01 | Figure 3

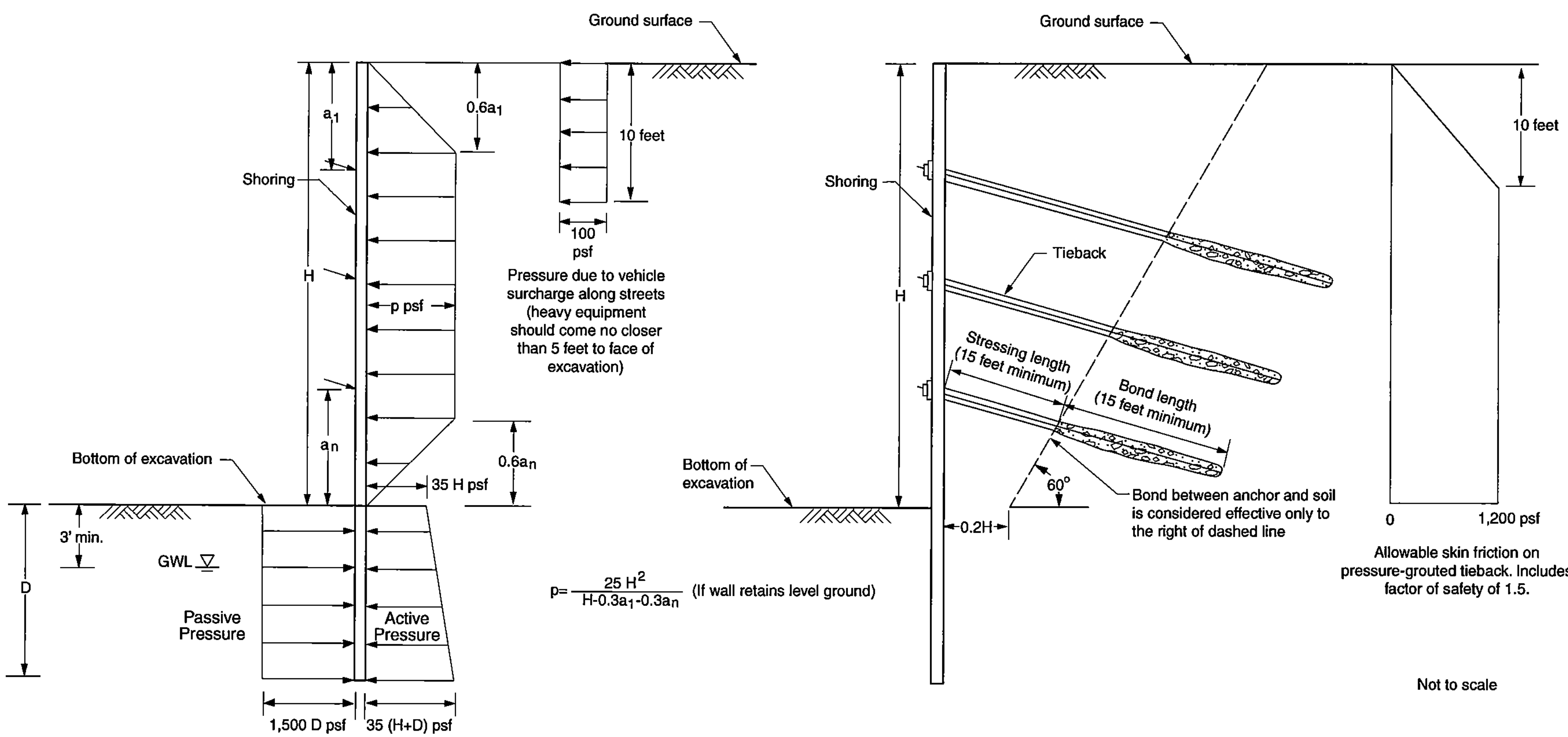
I	**Not felt by people, except under especially favorable circumstances. However, dizziness or nausea may be experienced.** Sometimes birds and animals are uneasy or disturbed. Trees, structures, liquids, bodies of water may sway gently, and doors may swing very slowly.
II	**Felt indoors by a few people, especially on upper floors of multi-story buildings, and by sensitive or nervous persons.** As in Grade I, birds and animals are disturbed, and trees, structures, liquids and bodies of water may sway. Hanging objects swing, especially if they are delicately suspended.
III	**Felt indoors by several people, usually as a rapid vibration that may not be recognized as an earthquake at first. Vibration is similar to that of a light, or lightly loaded trucks, or heavy trucks some distance away. Duration may be estimated in some cases.** Movements may be appreciable on upper levels of tall structures. Standing motor cars may rock slightly.
IV	**Felt indoors by many, outdoors by a few. Awakens a few individuals, particularly light sleepers, but frightens no one except those apprehensive from previous experience. Vibration like that due to passing of heavy, or heavily loaded trucks. Sensation like a heavy body striking building, or the falling of heavy objects inside.** Dishes, windows and doors rattle; glassware and crockery clink and clash. Walls and house frames creak, especially if intensity is in the upper range of this grade. Hanging objects often swing. Liquids in open vessels are disturbed slightly. Stationary automobiles rock noticeably.
V	**Felt indoors by practically everyone, outdoors by most people. Direction can often be estimated by those outdoors. Awakens many, or most sleepers. Frightens a few people, with slight excitement; some persons run outdoors.** Buildings tremble throughout. Dishes and glassware break to some extent. Windows crack in some cases, but not generally. Vases and small or unstable objects overturn in many instances, and a few fall. Hanging objects and doors swing generally or considerably. Pictures knock against walls, or swing out of place. Doors and shutters open or close abruptly. Pendulum clocks stop, or run fast or slow. Small objects move, and furnishings may shift to a slight extent. Small amounts of liquids spill from well-filled open containers. Trees and bushes shake slightly.
VI	**Felt by everyone, indoors and outdoors. Awakens all sleepers. Frightens many people; general excitement, and some persons run outdoors.** Persons move unsteadily. Trees and bushes shake slightly to moderately. Liquids are set in strong motion. Small bells in churches and schools ring. Poorly built buildings may be damaged. Plaster falls in small amounts. Other plaster cracks somewhat. Many dishes and glasses, and a few windows break. Knickknacks, books and pictures fall. Furniture overturns in many instances. Heavy furnishings move.
VII	**Frightens everyone. General alarm, and everyone runs outdoors.** People find it difficult to stand. Persons driving cars notice shaking. Trees and bushes shake moderately to strongly. Waves form on ponds, lakes and streams. Water is muddied. Gravel or sand stream banks cave in. Large church bells ring. Suspended objects quiver. Damage is negligible in buildings of good design and construction; slight to moderate in well-built ordinary buildings; considerable in poorly built or badly designed buildings, adobe houses, old walls (especially where laid up without mortar), spires, etc. Plaster and some stucco fall. Many windows and some furniture break. Loosened brickwork and tiles shake down. Weak chimneys break at the roofline. Cornices fall from towers and high buildings. Bricks and stones are dislodged. Heavy furniture overturns. Concrete irrigation ditches are considerably damaged.
VIII	**General fright, and alarm approaches panic.** Persons driving cars are disturbed. Trees shake strongly, and branches and trunks break off (especially palm trees). Sand and mud erupts in small amounts. Flow of springs and wells is temporarily and sometimes permanently changed. Dry wells renew flow. Temperatures of spring and well waters varies. Damage slight in brick structures built especially to withstand earthquakes; considerable in ordinary substantial buildings, with some partial collapse; heavy in some wooden houses, with some tumbling down. Panel walls break away in frame structures. Decayed pilings break off. Walls fall. Solid stone walls crack and break seriously. Wet grounds and steep slopes crack to some extent. Chimneys, columns, monuments and factory stacks and towers twist and fall. Very heavy furniture moves conspicuously or overturns.
IX	**Panic is general.** Ground cracks conspicuously. Damage is considerable in masonry structures built especially to withstand earthquakes; great in other masonry buildings - some collapse in large part. Some wood frame houses built especially to withstand earthquakes are thrown out of plumb, others are shifted wholly off foundations. Reservoirs are seriously damaged and underground pipes sometimes break.
X	**Panic is general.** Ground, especially when loose and wet, cracks up to widths of several inches; fissures up to a yard in width run parallel to canal and stream banks. Landsliding is considerable from river banks and steep coasts. Sand and mud shifts horizontally on beaches and flat land. Water level changes in wells. Water is thrown on banks of canals, lakes, rivers, etc. Dams, dikes, embankments are seriously damaged. Well-built wooden structures and bridges are severely damaged, and some collapse. Dangerous cracks develop in excellent brick walls. Most masonry and frame structures, and their foundations are destroyed. Railroad rails bend slightly. Pipe lines buried in earth tear apart or are crushed endwise. Open cracks and broad wavy folds open in cement pavements and asphalt road surfaces.
XI	**Panic is general.** Disturbances in ground are many and widespread, varying with the ground material. Broad fissures, earth slumps, and land slips develop in soft, wet ground. Water charged with sand and mud is ejected in large amounts. Sea waves of significant magnitude may develop. Damage is severe to wood frame structures, especially near shock centers, great to dams, dikes and embankments, even at long distances. Few if any masonry structures remain standing. Supporting piers or pillars of large, well-built bridges are wrecked. Wooden bridges that "give" are less affected. Railroad rails bend greatly and some thrust endwise. Pipe lines buried in earth are put completely out of service.
XII	**Panic is general.** Damage is total, and practically all works of construction are damaged greatly or destroyed. Disturbances in the ground are great and varied, and numerous shearing cracks develop. Landslides, rock falls, and slumps in river banks are numerous and extensive. Large rock masses are wrenched loose and torn off. Fault slips develop in firm rock, and horizontal and vertical offset displacements are notable. Water channels, both surface and underground, are disturbed and modified greatly. Lakes are dammed, new waterfalls are produced, rivers are deflected, etc. Surface waves are seen on ground surfaces. Lines of sight and level are distorted. Objects are thrown upward into the air.

SAN TOMAS BUSINESS PARK
Santa Clara, California

MODIFIED MERCALLI INTENSITY SCALE

Treadwell&Rollo

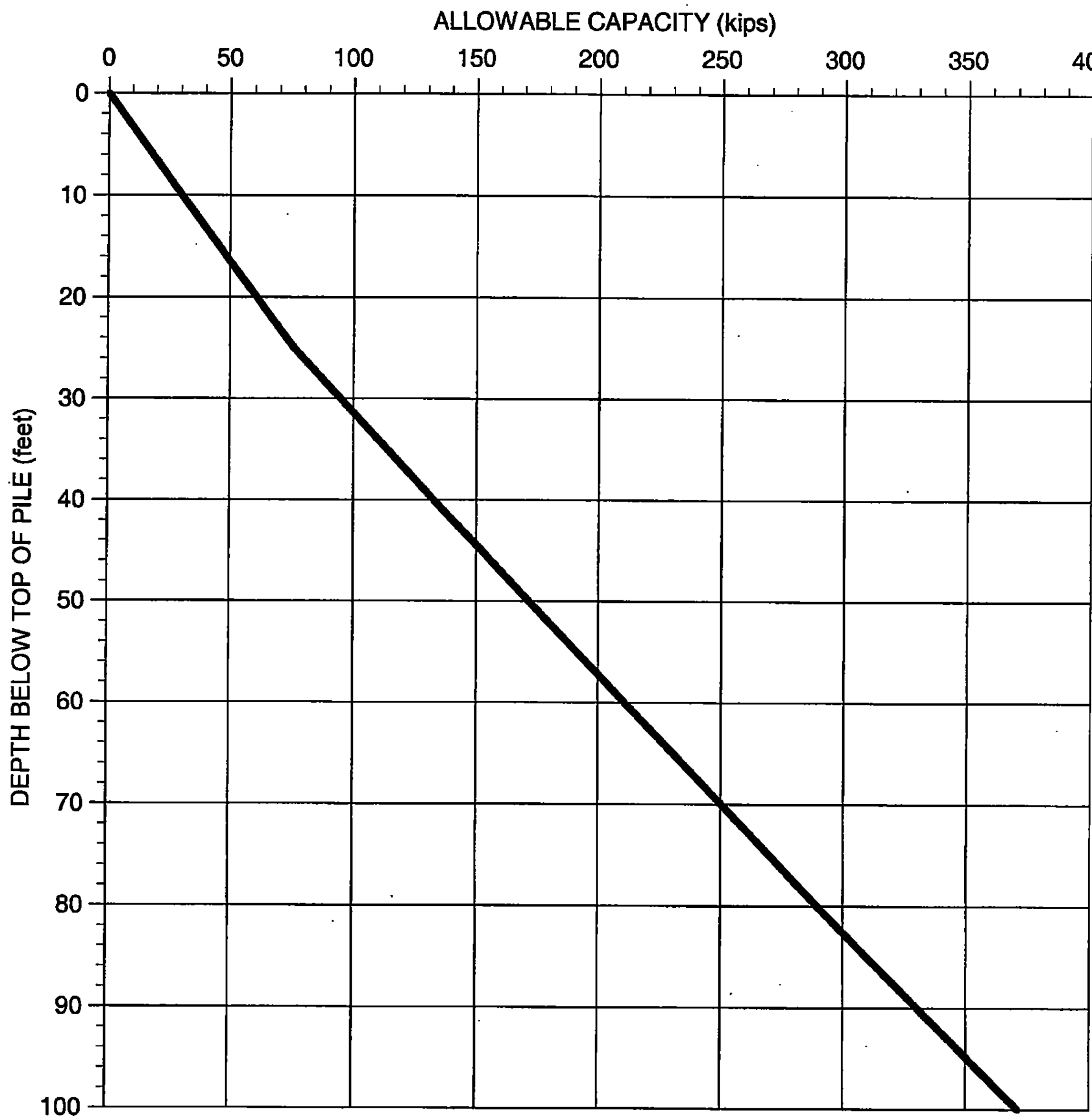
Date 05/20/08 | Project No. 4783.01 | Figure 4



Notes:
1. Passive pressure includes a factor of safety of about 1.5.
2. For soldier piles spaced at more than three times the soldier pile diameter, the passive pressure should be assumed to act over three diameters.
3. Active pressure should be assumed to act over one pile diameter.

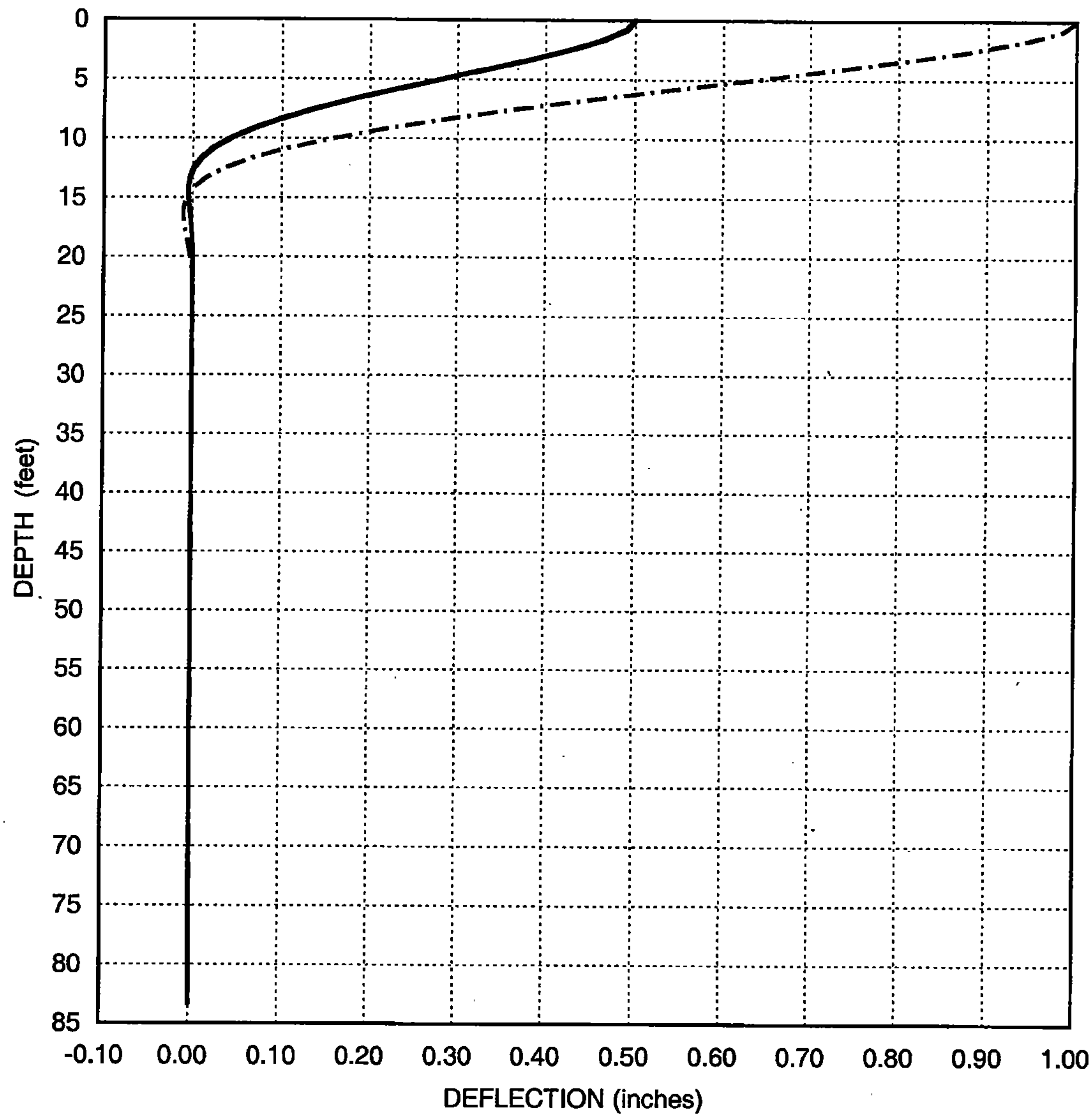
SAN TOMAS BUSINESS PARK Santa Clara, California		
DESIGN PARAMETERS FOR SOLDIER-PILE-AND-LAGGING TEMPORARY SHORING SYSTEM		
Date 05/20/08	Project No. 4783.01	Figure 5

Treadwell&Rollo



Notes: 1. The indicated capacities are for dead plus live loads. For wind and seismic loading, the indicated capacities can be increased by one-third. For temporary uplift load, use the allowable capacity curve.
2. Capacities are based on the allowable strength of the supporting soil; the structural capacity of the pile may govern.
3. Piles should be spaced no closer than three times the width center to center.
4. Pile capacities are based on skin friction only.

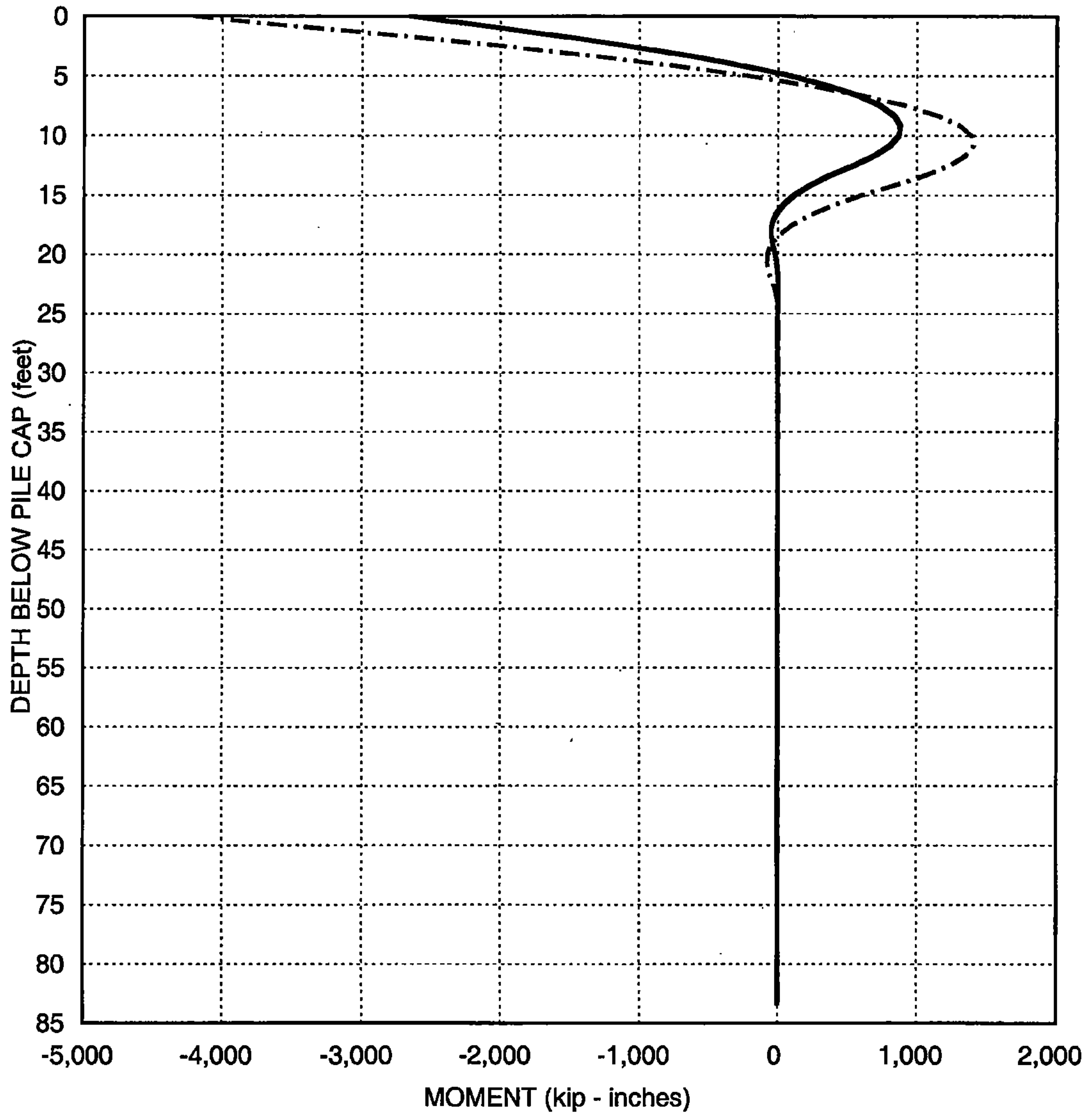
SAN TOMAS BUSINESS PARK Santa Clara, California	ALLOWABLE COMPRESSION CAPACITY FOR 14-INCH PRECAST PRESTRESSED, CONCRETE PILES		
Treadwell&Rollo	Date 12/19/07	Project No. 4783.01	Figure 6



Curve	Restraint	Lateral Load, H (kips)
——— (solid)	Fixed	60
—·—·—·— (dash-dot)	Fixed	83

Notes: 1. Assumes center to center spacing of piles is at least 6 diameters.
2. Assumes there is no applied moment at the pile head.

SAN TOMAS BUSINESS PARK Santa Clara, California	DEFLECTION PROFILES FOR 14-INCH SQUARE PRECAST, PRESTRESSED CONCRETE PILES, FIXED HEAD CONDITION
Treadwell&Rollo	Date 05/29/08 | Project No. 4783.01 | Figure 7



Curve	Restraint	Lateral Load, H (kips)	Deflection at Pile Head (in)
	Fixed	60	0.5
	Fixed	83	1.0

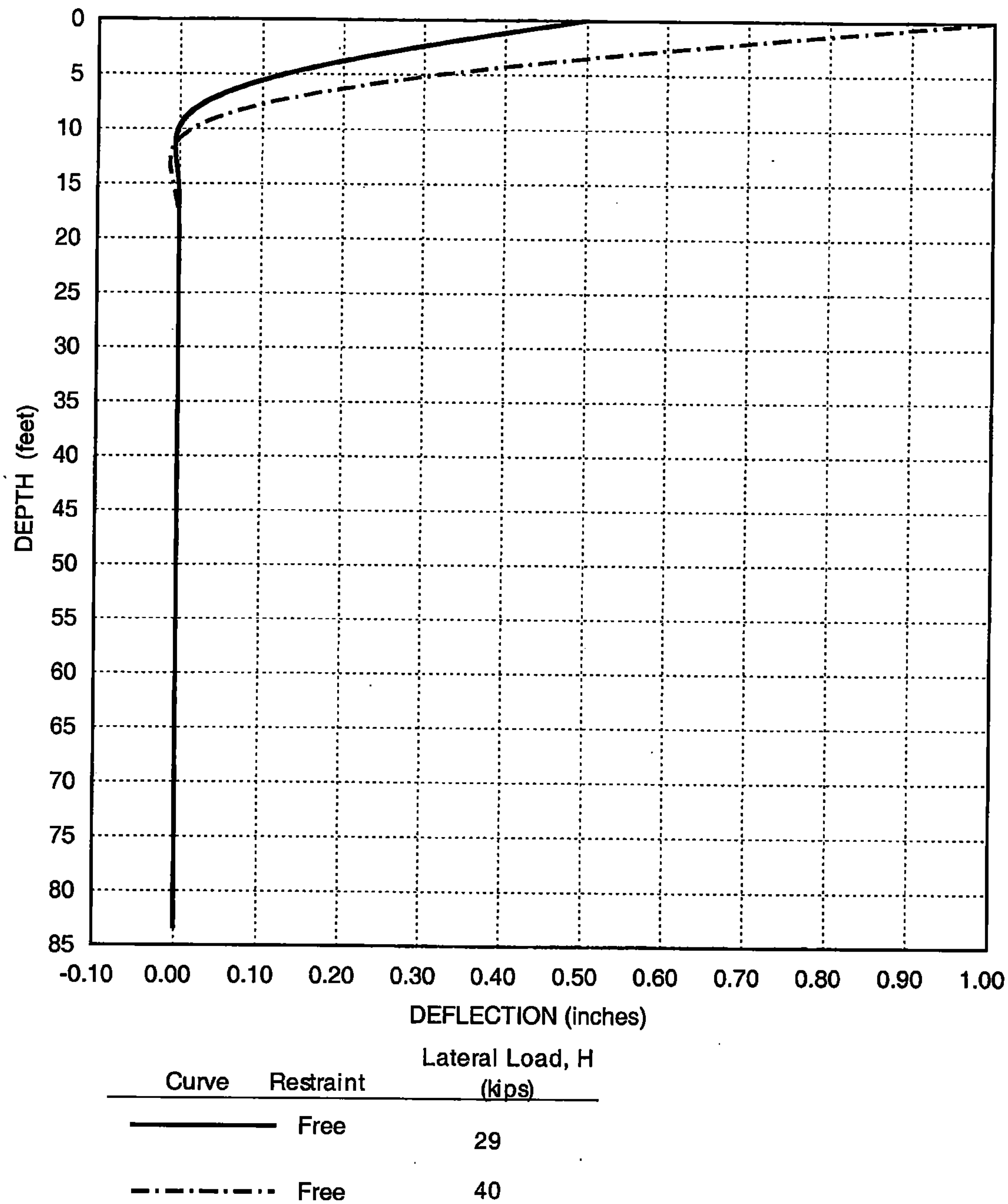
Notes: 1. Assumes center to center spacing of piles is at least 6 diameters.
2. Assumes there is no applied moment at the pile top.

SAN TOMAS BUSINESS PARK
City, California

Treadwell&Rollo

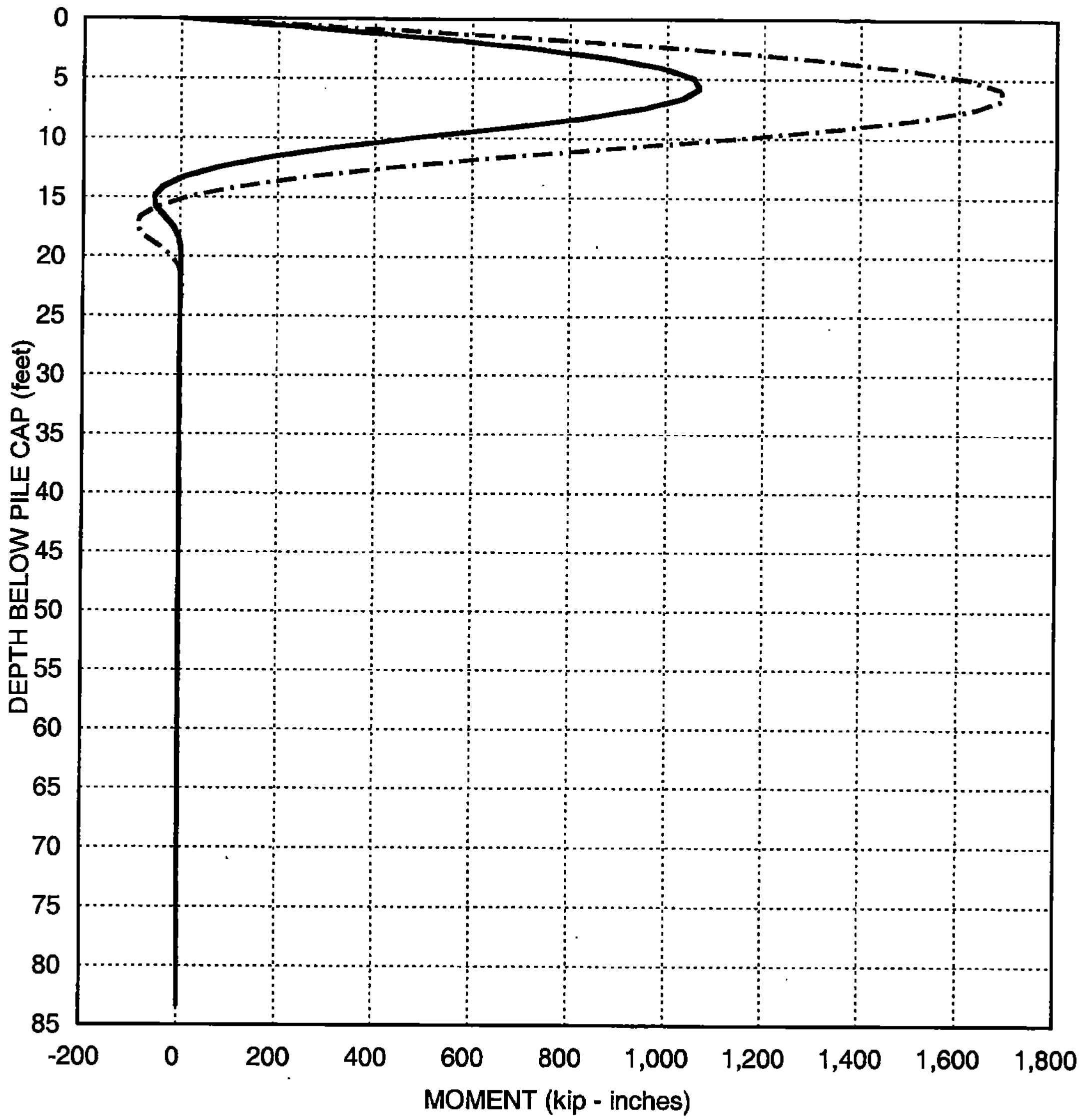
MOMENT PROFILES FOR 14-INCH SQUARE PRECAST, PRESTRESSED CONCRETE PILES, FIXED HEAD CONDITION

Date 05/29/08	Project No. 4783.01	Figure 8



Notes: 1. Assumes center to center spacing of piles is at least 6 diameters.
2. Assumes there is no applied moment at the pile head.

SAN TOMAS BUSINESS PARK Santa Clara, California	**DEFLECTION PROFILES FOR 14-INCH SQUARE PRECAST, PRESTRESSED CONCRETE PILES, FREE HEAD CONDITION**
Treadwell&Rollo	Date 05/29/08 · Project No. 4783.01 · Figure 9



Curve	Restraint	Lateral Load, H (kips)	Deflection at Pile Head (in)
————	Free	29	0.5
—·—·—·—	Free	40	1.0

Notes: 1. Assumes center to center spacing of piles is at least 6 diameters.
2. Assumes there is no applied moment at the pile top.

SAN TOMAS BUSINESS PARK City, California	DEFLECTION PROFILES FOR 14-INCH SQUARE PRECAST, PRESTRESSED CONCRETE PILES, FREE HEAD CONDITION
Treadwell&Rollo	Date 05/29/08 | Project No. 4783.01 | Figure 10

APPENDIX A

Logs of Test Borings

PROJECT:	SAN TOMAS BUSINESS PARK Santa Clara, California	Log of Boring B-1 PAGE 1 OF 2

Boring location: See Site Plan, Figure 2

Logged by: M. Colombo

Date started: 3/5/08 | Date finished: 3/5/08

Drilling method: Rotary Wash

Hammer weight/drop: 140 lbs./30-inches | Hammer type: Automatic Hammer

Sampler: Sprague & Henwood (S&H), Standard Penetration Test (SPT), Shelby Tube (ST)

DEPTH (feet)	SAMPLES: Sampler Type	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION Ground Surface Elevation: 42 feet[2]	LABORATORY TEST DATA: Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
0–2					2.5-inches thick Asphalt Concrete (AC) 21.5-inches thick Aggregate Base (AB)						
3	S&H	9, 10, 15	18	CH	SANDY CLAY (CH) dark brown to black, very stiff, moist, fine- to coarse-grained sand LL = 53, PI = 30						
6	S&H	8, 11, 17	20	CL	CLAY with SAND (CL) yellow-brown with mottled olive-brown, very stiff, moist, fine-grained sand						
9	ST										
10	SPT	13, 18, 29	56	SC	CLAYEY SAND with GRAVEL (SC) yellow-brown, very dense, wet, medium- to coarse-grained sand, fine-grained gravel						
13					∇ 3/6/07, 8:30 am						
15	S&H	4, 5, 6	8	CL	CLAY with SAND (CL) light yellow-brown, medium stiff to stiff, wet, very fine-grained sand, high silt content					26.4	99
20	ST		150 to 200 psi		Consolidation Test, see Figure C-1	TxUU	1,800	1,390		26.5	99
21.5										25.8	96
25	SPT	2, 5, 3	10	CL	SANDY CLAY (CL) gray, stiff, wet, fine-grained sand, with thin sand interbeds						
27					grades less sandy, olive-gray						
30	S&H	5	11								

Treadwell&Rollo

Project No.: 4783.01 | Figure: A-1a

TEST GEOTECH LOG 478301.GPJ TR.GDT 6/5/08

PROJECT:	SAN TOMAS BUSINESS PARK Santa Clara, California	Log of Boring B-1

DEPTH (feet)	Sampler Type	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
30–31	S&H	8 8	11	CL	SANDY CLAY (CL) (continued)					21.7	103
34–36	ST		150 to 200 psi	CL							
39–41	S&H	10 21 23	31	SC	CLAYEY SAND (SC) gray, dense, wet, fine-grained sand				29.1	18.5	112
41				CL	SANDY CLAY (CL) gray, very stiff, moist, fine-grained sand, with trace gravel						
44–45.5	S&H		23	CL							
49–50.5	SPT		22	CL							

Boring terminated at a depth of 50.5 feet below ground surface.
Boring backfilled with cement grout.
Groundwater encountered at a depth of 12.8 feet during drilling.

[1] S&H and SPT blow counts converted to N-Values using a factor of 0.7 and 1.2, respectively.
[2] Elevations based on a topographic survey by Kier & Wright Civil Engineers & Surveyors, dated Janurary 2006

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Project No.: 4783.01

Figure: A-1b

TEST GEOTECH LOG 478301.GPJ TR.GDT 6/5/08

PROJECT:	SAN TOMAS BUSINESS PARK Santa Clara, California	Log of Boring B-2

Boring location:	See Site Plan, Figure 2
Date started:	3/6/08
Date finished:	3/6/08
Drilling method:	Rotary Wash
Hammer weight/drop:	140 lbs./30-inches
Hammer type:	Automatic Hammer
Sampler:	Sprague & Henwood (S&H), Standard Penetration Test (SPT), Shelby Tube (ST)
Logged by:	M. Colombo

DEPTH (feet)	Sampler Type	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION (Ground Surface Elevation: 41.5 feet[2])	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
0					2-inches thick Asphalt Concrete (AC)						
0–1					5-inches thick Aggregate Base (AB)						
1–14				CL	CLAY (CL) yellow-brown with mottled olive-brown, very stiff, moist, with sand, high silt content						
5–6	S&H	7 10 17	19								
9–10	ST		100 to 200 psi							17.2	110
13					grades medium stiff to stiff						
14–19.5				CL	SANDY CLAY (CL) yellow-brown with mottled iron stain, medium stiff to stiff, wet, very fine- to fine-grained sand						
14–15	S&H	3 5 6	8							26.2	123
19–20	ST		100 to 200 psi						59.9	23.3	97
19.5–22				SP	SAND (SP) yellow-brown, medium dense, wet, fine-grained sand						
22–28				CL	SANDY CLAY (CL) dark olive-gray, medium stiff, wet, fine-grained sand						
24–25	S&H	3 5 6	8							21.4	104
28–30				SC	CLAYEY SAND with GRAVEL (SC) olive-gray, dense, wet, medium- to coarse-grained sand, fine-grained gravel						
29–30	SPT	18	48								

LABORATORY TEST DATA

TEST GEOTECH LOG 478301.GPJ TR.GDT 6/5/08

Treadwell&Rollo

Project No.: 4783.01

Figure: A-2a

PROJECT:	SAN TOMAS BUSINESS PARK Santa Clara, California	Log of Boring B-2 PAGE 2 OF 2

DEPTH (feet)	Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
30–31	SPT		21 19	48	SC	CLAYEY SAND with GRAVEL (SC) (continued)						
32–37					CL	CLAY with SAND (CL) olive-brown, stiff to very stiff, moist, fine-grained sand						
34–35	S&H		8 9 13	15								
37–42					CL	SILT with SAND (ML) olive-brown, very stiff, wet, fine-grained sand, high silt content						
39–40	SPT		3 6 10	19		LL = 27, PI = 2				75.1		
42–50.5					CL	grades less sandy, olive-gray SANDY CLAY (CL) olive-brown, very stiff, wet, fine-grained sand, high clay content						
44–45	S&H		8 13 15	20								
49–50	SPT		5 8 8	19								

Boring terminated at a depth of 50.5 feet below ground surface.
Boring backfilled with cement grout.
Groundwater level obscured by rotary wash drilling method.

[1] S&H and SPT blow counts converted to N-Values using a factor of 0.7 and 1.2, respectively.
[2] Elevations based on a topographic survey by Kier & Wright Civil Engineers & Surveyors, dated Janurary 2006

Treadwell&Rollo

Project No.: 4783.01 | Figure: A-2b

TEST GEOTECH LOG 478301.GPJ TR.GDT 6/5/08

PROJECT:	SAN TOMAS BUSINESS PARK Santa Clara, California	**Log of Boring B-3**

Boring location: See Site Plan, Figure 2

Logged by: M. Colombo

Date started: 3/5/08 | Date finished: 3/5/08

Drilling method: Rotary Wash

Hammer weight/drop: 140 lbs./30-inches | Hammer type: Automatic Hammer

Sampler: Sprague & Henwood (S&H), Standard Penetration Test (SPT), Shelby Tube (ST)

Ground Surface Elevation: 42.5 feet[2]

DEPTH (feet)	SAMPLES: Sampler Type	SAMPLES: Blows/ 6"	SAMPLES: SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA: Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
0–1					2-inches thick Asphalt Concrete (AC)						
					5-inches thick Aggregate Base (AB)						
1–4				CL	CLAY with SAND (CL) dark brown to black, very stiff, moist, with trace roots						
3–4	S&H	6 10 13	16		LL = 37, PI = 16					17.4	107
4–10				CL	CLAY with SAND (CL) yellow-brown, medium stiff to stiff, moist						
5–6	S&H	6 7 5	8							16.1	111
9–11	ST										
10–13.5				SM	SILTY SAND (SM) yellow-brown, mottled iron stain, loose, wet, fine-grained sand						
12–13.5	S&H	4 4 4	6		PI = NP				41.7	23.9	100
13.5–22				CL	CLAY with SAND (CL) yellow-brown, mottled iron stain, medium stiff, wet, very fine-grained sand, high silt content						
14–15.5	SPT	1 3 3	7								
19–20.5	ST		100 to 200 psi		grades stiff	TxUU	1,800	1,810		28.1	97
22–30				CL	SANDY CLAY (CL) light olive-gray with mottled olive-brown, stiff, wet, fine-grained sand, high silt content						
24–25.5	ST		100 to 200 psi								
26–27.5	S&H	6 6 8	10								
29–30	SPT	4	14								

Treadwell&Rollo

Project No.: 4783.01 | Figure: A-3a

PROJECT: **SAN TOMAS BUSINESS PARK**
Santa Clara, California

Log of Boring B-3

DEPTH (feet)	Sampler Type	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
30–31	SPT	4, 8	14	CL	SANDY CLAY (CL) (continued) color change to olive-brown, sandier						
34–35	S&H	5, 9, 10	13	CL							
38.5				SM/ ML	SILTY SAND (SM)/ SANDY SILT (ML) olive-gray, medium dense, very stiff, wet, fine- to coarse-grained sand, with clayey interbeds, scattered fine gravel interbeds PI = NP						
39–40	S&H	17, 11, 16	19	SM/ ML					38.1	19.7	109
44–45	SPT	5, 7, 11	22	SM/ ML							
46.5				CL	CLAY (CL) olive-gray and olive brown mottled, very stiff, moist, trace fine-grained sand						
49–50	S&H	10, 17, 24	29	CL							

Depth scale: 31–60 feet.

Boring terminated at a depth of 50.5 feet below ground surface.
Boring backfilled with cement grout.
Groundwater level obscured by rotary wash drilling method.

[1] S&H and SPT blow counts converted to N-Values using a factor of 0.7 and 1.2, respectively.
[2] Elevations based on a topographic survey by Kier & Wright Civil Engineers & Surveyors, dated Janurary 2006

Treadwell&Rollo

Project No.: 4783.01 | Figure: A-3b

TEST GEOTECH LOG 478301.GPJ TR.GDT 6/5/08

UNIFIED SOIL CLASSIFICATION SYSTEM

Major Divisions		Symbols	Typical Names
Coarse-Grained Soils (more than half of soil > no. 200 sieve size	Gravels (More than half of coarse fraction > no. 4 sieve size)	GW	Well-graded gravels or gravel-sand mixtures, little or no fines
		GP	Poorly-graded gravels or gravel-sand mixtures, little or no fines
		GM	Silty gravels, gravel-sand-silt mixtures
		GC	Clayey gravels, gravel-sand-clay mixtures
	Sands (More than half of coarse fraction < no. 4 sieve size)	SW	Well-graded sands or gravelly sands, little or no fines
		SP	Poorly-graded sands or gravelly sands, little or no fines
		SM	Silty sands, sand-silt mixtures
		SC	Clayey sands, sand-clay mixtures
Fine -Grained Soils (more than half of soil < no. 200 sieve size)	Silts and Clays LL = < 50	ML	Inorganic silts and clayey silts of low plasticity, sandy silts, gravelly silts
		CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, lean clays
		OL	Organic silts and organic silt-clays of low plasticity
	Silts and Clays LL = > 50	MH	Inorganic silts of high plasticity
		CH	Inorganic clays of high plasticity, fat clays
		OH	Organic silts and clays of high plasticity
Highly Organic Soils		PT	Peat and other highly organic soils

GRAIN SIZE CHART

Classification	Range of Grain Sizes: U.S. Standard Sieve Size	Range of Grain Sizes: Grain Size in Millimeters
Boulders	Above 12"	Above 305
Cobbles	12" to 3"	305 to 76.2
Gravel	3" to No. 4	76.2 to 4.76
coarse	3" to 3/4"	76.2 to 19.1
fine	3/4" to No. 4	19.1 to 4.76
Sand	No. 4 to No. 200	4.76 to 0.075
coarse	No. 4 to No. 10	4.76 to 2.00
medium	No. 10 to No. 40	2.00 to 0.420
fine	No. 40 to No. 200	0.420 to 0.075
Silt and Clay	Below No. 200	Below 0.075

Unstabilized groundwater level

Stabilized groundwater level

SAMPLE DESIGNATIONS/SYMBOLS

Sample taken with Sprague & Henwood split-barrel sampler with a 3.0-inch outside diameter and a 2.43-inch inside diameter. Darkened area indicates soil recovered

Classification sample taken with Standard Penetration Test sampler

Undisturbed sample taken with thin-walled tube

Disturbed sample

Sampling attempted with no recovery

Core sample

Analytical laboratory sample

Sample taken with Direct Push sampler

Sonic

SAMPLER TYPE

C Core barrel

CA California split-barrel sampler with 2.5-inch outside diameter and a 1.93-inch inside diameter

D&M Dames & Moore piston sampler using 2.5-inch outside diameter, thin-walled tube

O Osterberg piston sampler using 3.0-inch outside diameter, thin-walled Shelby tube

PT Pitcher tube sampler using 3.0-inch outside diameter, thin-walled Shelby tube

S&H Sprague & Henwood split-barrel sampler with a 3.0-inch outside diameter and a 2.43-inch inside diameter

SPT Standard Penetration Test (SPT) split-barrel sampler with a 2.0-inch outside diameter and a 1.5-inch inside diameter

ST Shelby Tube (3.0-inch outside diameter, thin-walled tube) advanced with hydraulic pressure

SAN TOMAS BUSINESS PARK
Santa Clara, California

Treadwell&Rollo

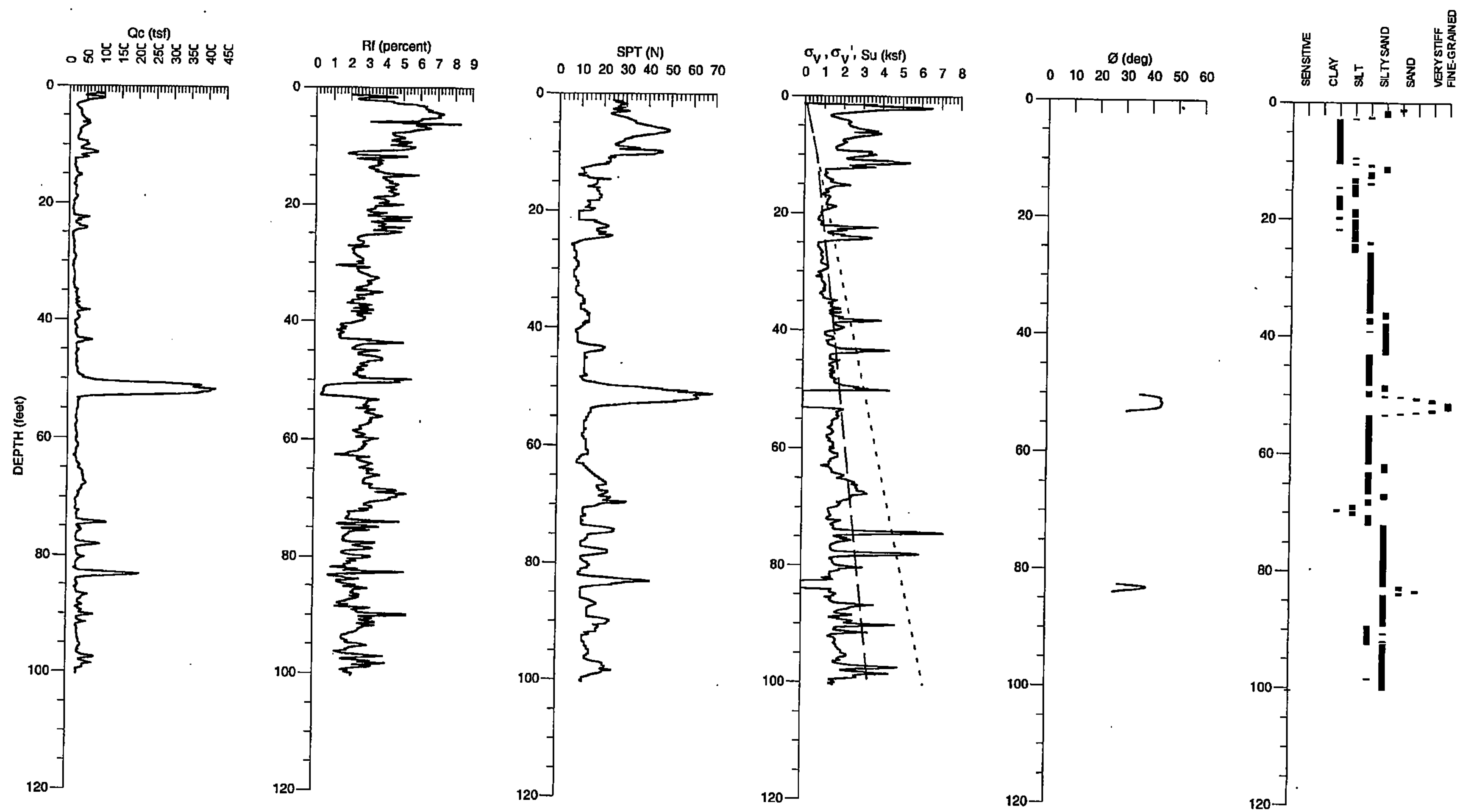
CLASSIFICATION CHART

Date 05/20/08 | Project No. 4783.01 | Figure A-4

Treadwell&Rollo

APPENDIX B

Logs of Cone Penetration Tests



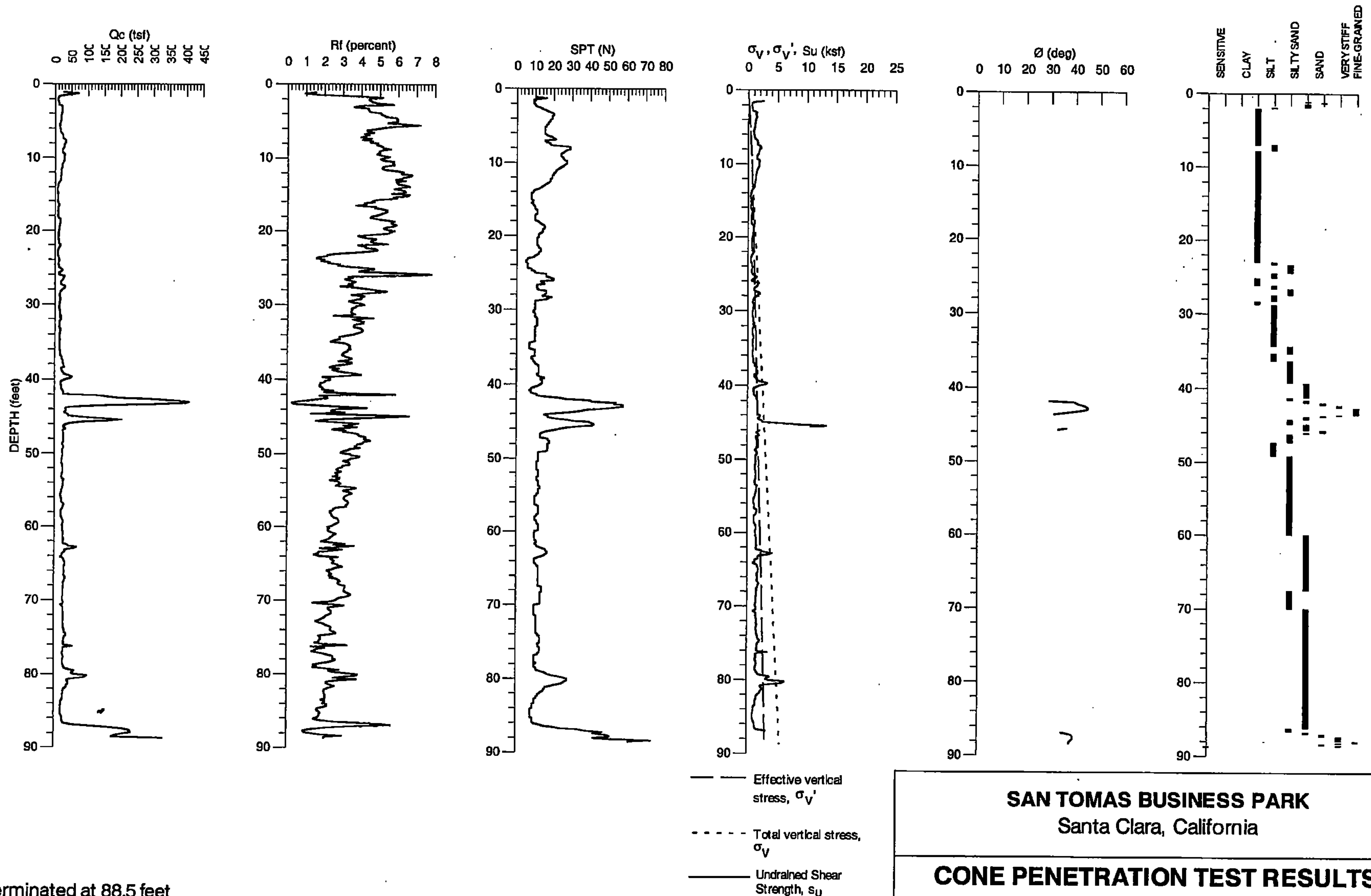
Terminated at 100.4 feet
Date performed: 03/06/08.
Ground surface elevation: 40.5 feet (MSL datum), Survey by Kier & Wright Civil Engineers & Surveyors, dated January 2008.

SAN TOMAS BUSINESS PARK
Santa Clara, California

CONE PENETRATION TEST RESULTS
CPT-1

Date 05/20/08	Project No. 4783.01	Figure B-1

Treadwell&Rollo



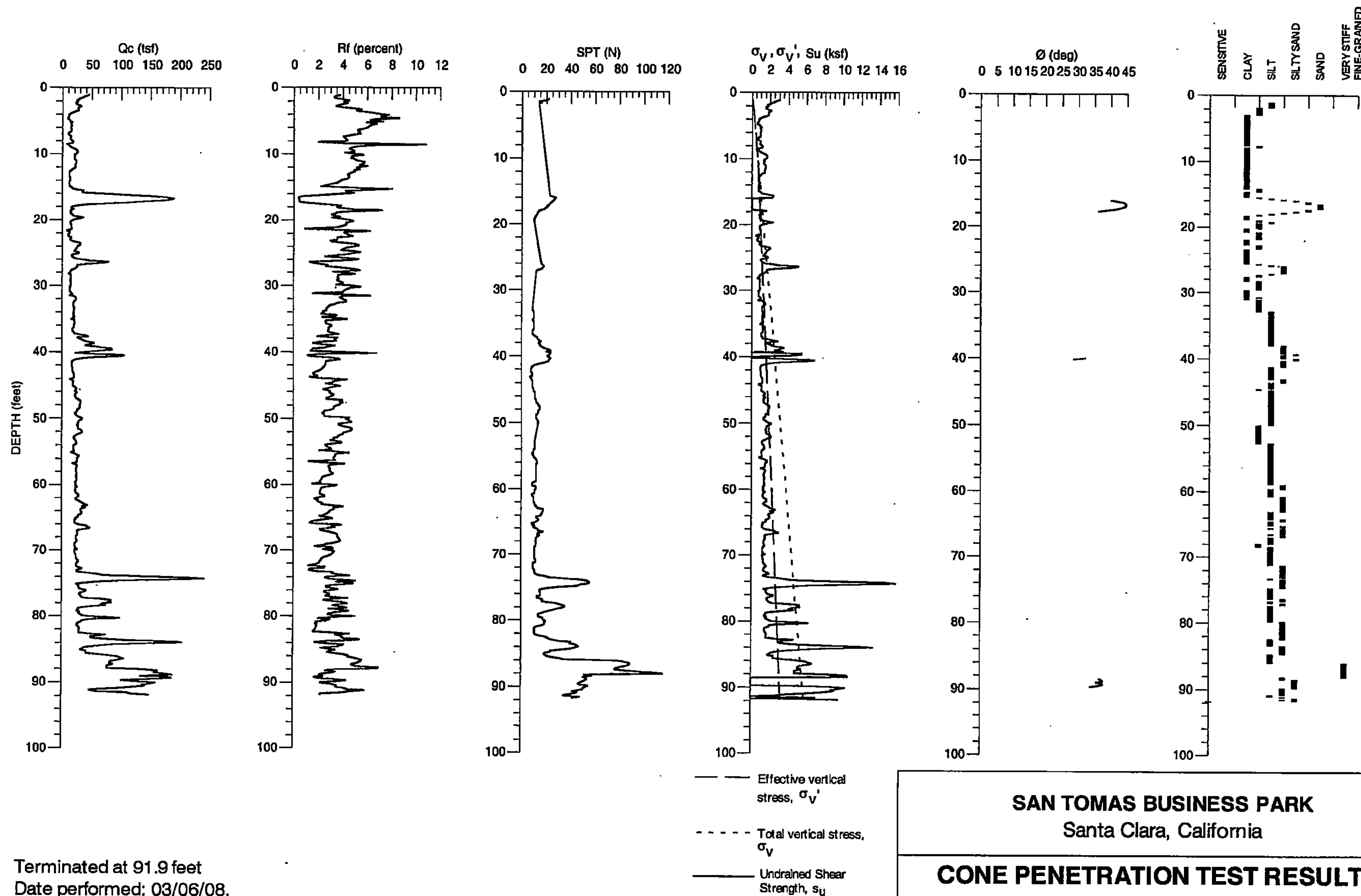
Terminated at 88.5 feet
Date performed: 03/06/08.
Ground surface elevation: 41 feet (MSL datum), Survey by Kier & Wright Civil Engineers & Surveyors, dated January 2008.

SAN TOMAS BUSINESS PARK
Santa Clara, California

CONE PENETRATION TEST RESULTS
CPT-2

Date 05/20/08	Project No. 4783.01	Figure B-2

Treadwell&Rollo



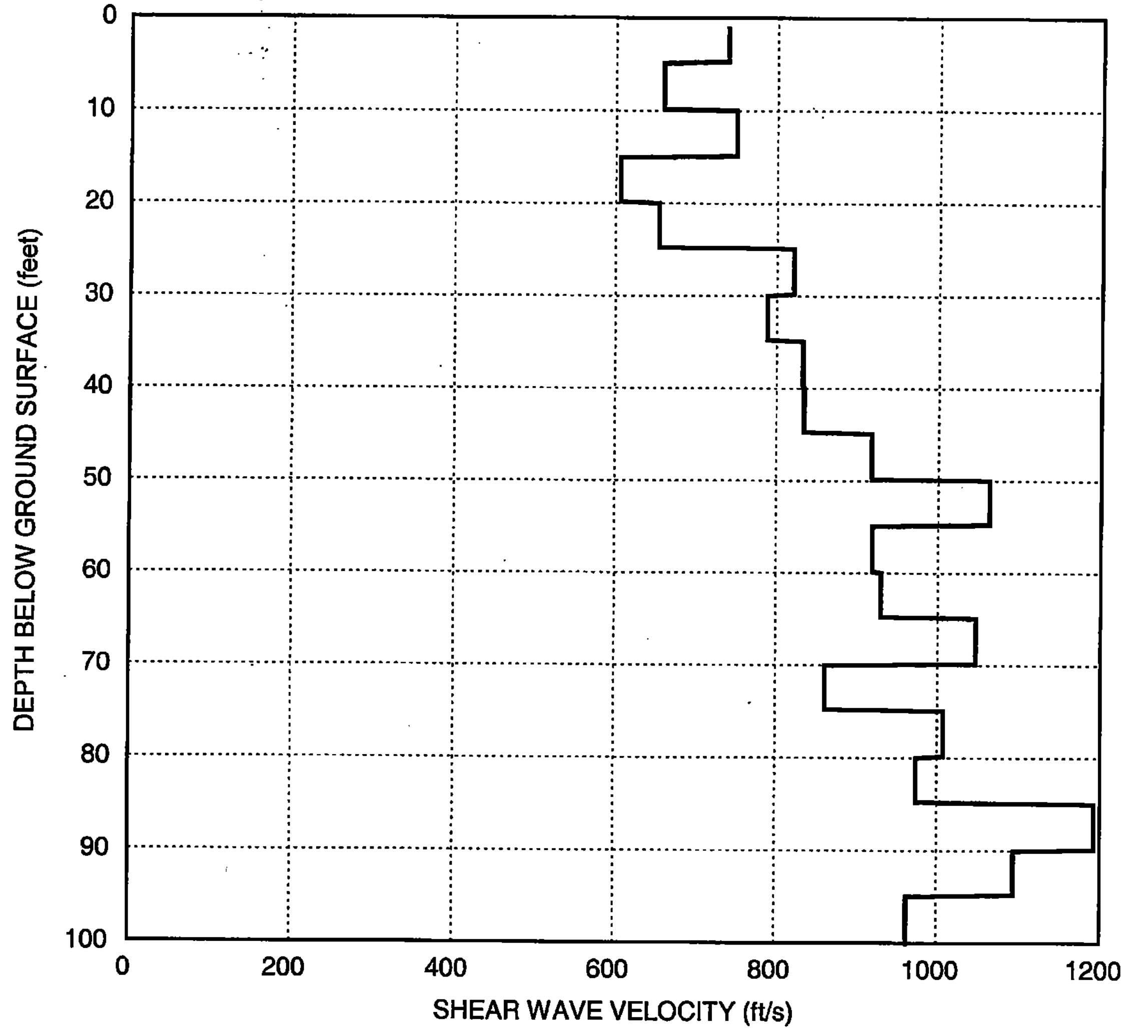
Terminated at 91.9 feet
Date performed: 03/06/08.
Ground surface elevation: 41 feet (MSL datum), Survey by Kier & Wright Civil Engineers & Surveyors, dated January 2008.

SAN TOMAS BUSINESS PARK
Santa Clara, California

CONE PENETRATION TEST RESULTS
CPT-3

Date 05/20/08	Project No. 4783.01	Figure B-3

Treadwell&Rollo

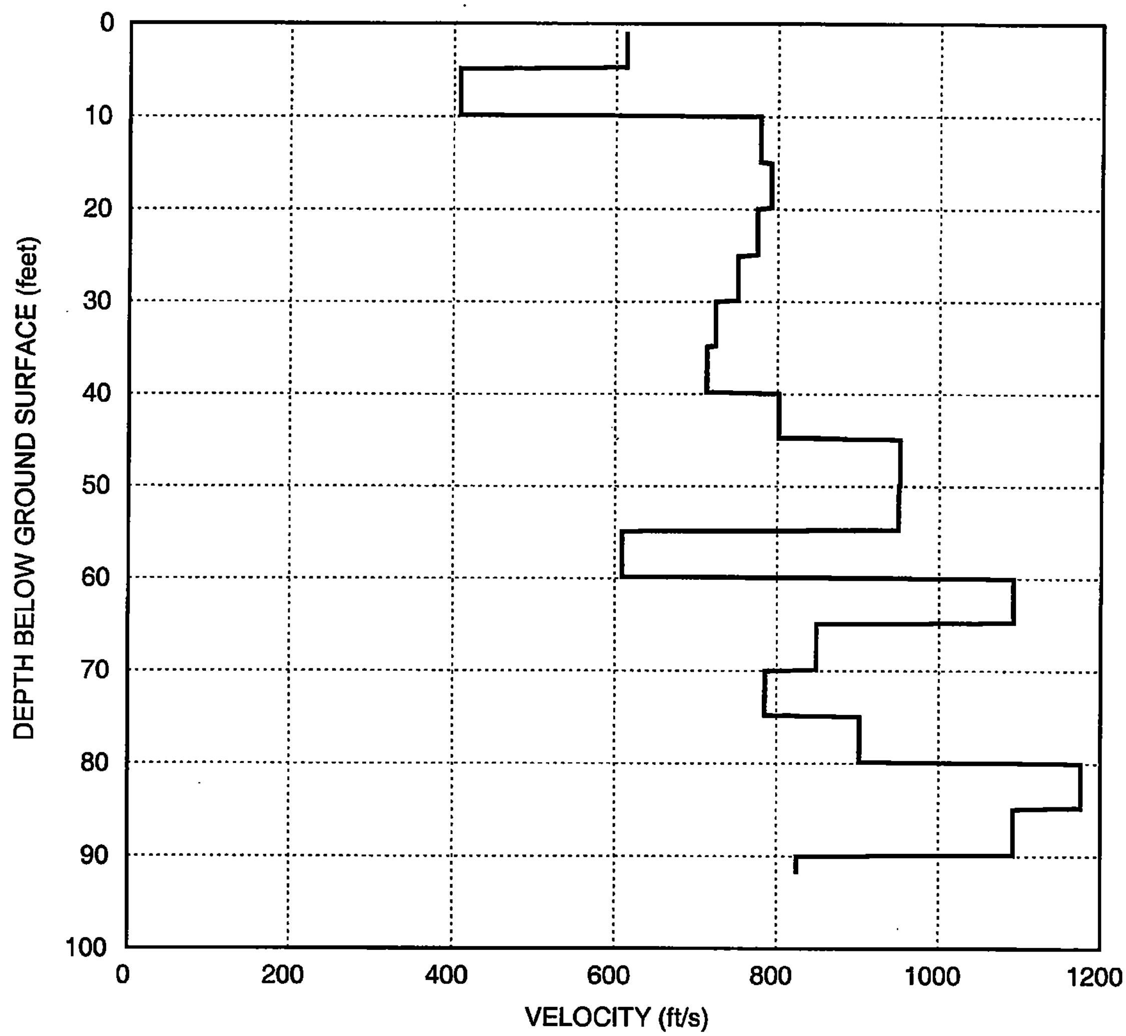


SAN TOMAS BUSINESS PARK
San Francisco, California

Treadwell&Rollo

SHEAR WAVE VELOCITY PROFILE
CPT-1

Date 05/29/08	Project No. 4783.01	Figure B-4

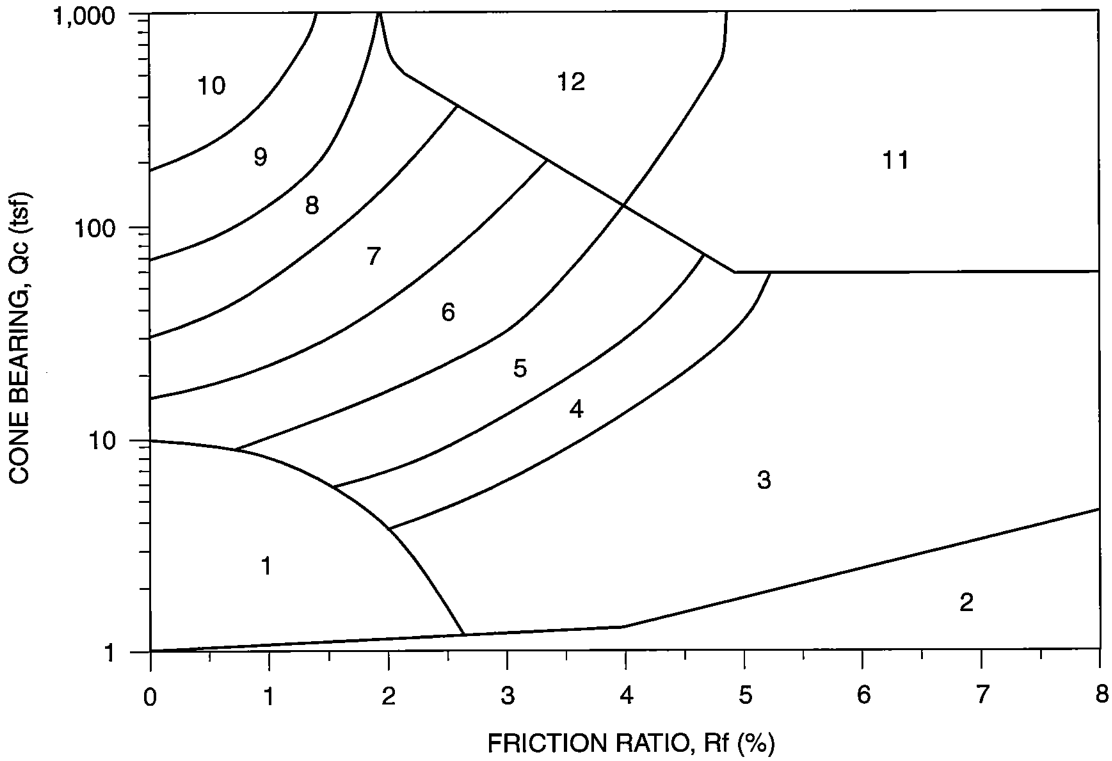


SAN TOMAS BUSINESS PARK
San Francisco, California

Treadwell&Rollo

SHEAR WAVE VELOCITY PROFILE
CPT-3

Date 05/29/08	Project No. 4783.01	Figure B-5



ZONE	Qc/N[1]	Su Factor (Nk)[2]	SOIL BEHAVIOR TYPE[1]
1	2	15 (10 for Qc ≤ 9 tsf)	Sensitive Fine-Grained
2	1	15 (10 for Qc ≤ 9 tsf)	Organic Material
3	1	15 (10 for Qc ≤ 9 tsf)	CLAY
4	1.5	15	SILTY CLAY to CLAY
5	2	15	CLAYEY SILT to SILTY CLAY
6	2.5	15	SANDY SILT to CLAYEY SILT
7	3	---	SILTY SAND to SANDY SILT
8	4	---	SAND to SILTY SAND
9	5	---	SAND
10	6	---	GRAVELLY SAND to SAND
11	1	15	Very Stiff Fine-Grained (*)
12	2	---	SAND to CLAYEY SAND (*)

(*) Overconsolidated or Cemented
Qc = Tip Bearing
Fs = Sleeve Friction
Rf = Fs/Qc x 100 = Friction Ratio

Note: Testing performed in accordance with ASTM D3441.

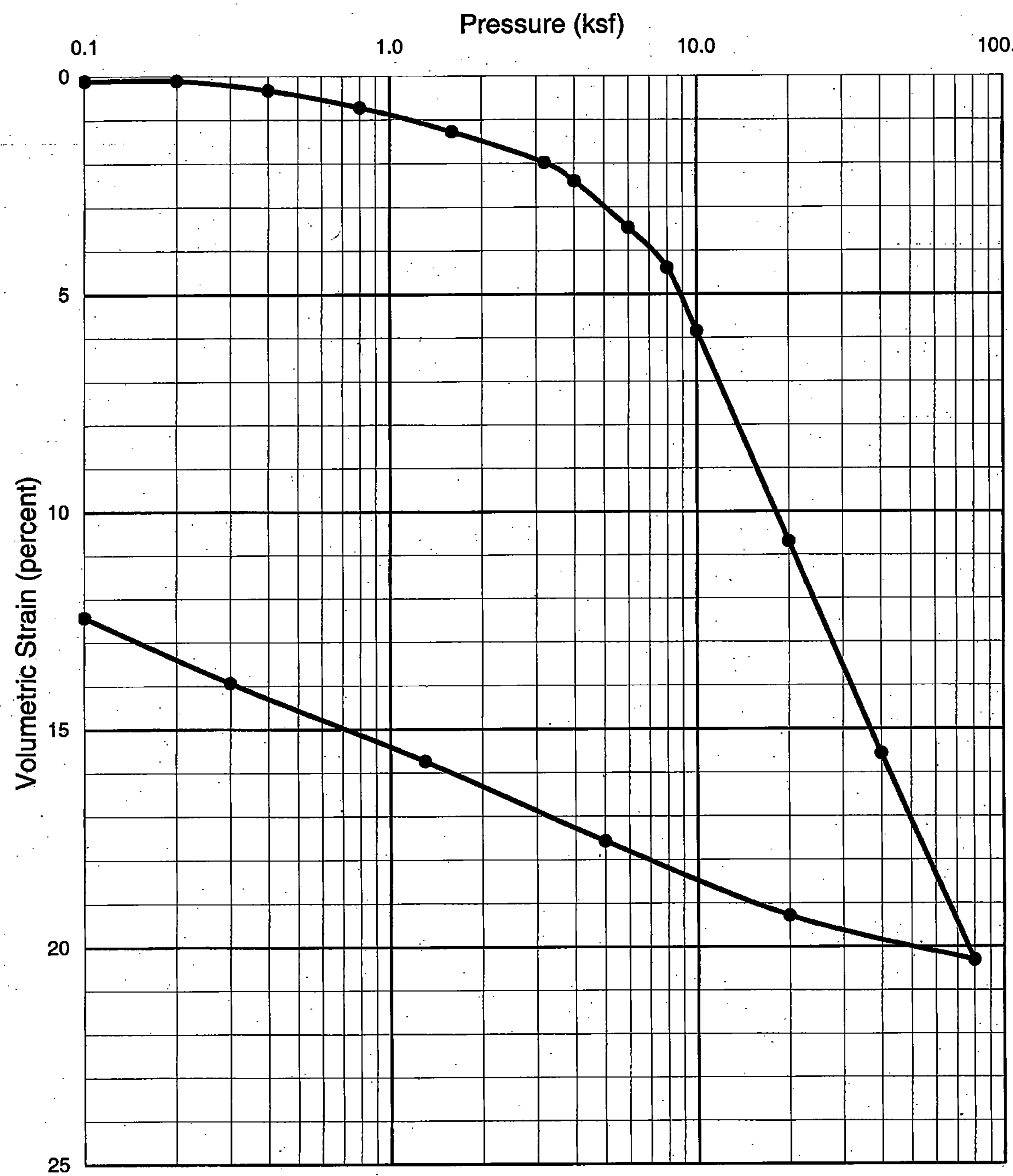
References: 1. Robertson, 1986, Olsen, 1988.
2. Bonaparte & Mitchell, 1979 (young Bay Mud Qc ≤9).
Estimated from local experience (fine-grained soils Qc > 9).

SAN TOMAS BUSINESS PARK Santa Clara, California	**CLASSIFICATION CHART FOR CONE PENETRATION TESTS**
Treadwell&Rollo	Date 05/20/08 | Project No. 4783.01 | Figure B-6

SARIMIENTO

APPENDIX C

Laboratory Test Results



Sampler Type: Shelby Tube				Condition	Before Test		After Test	
Diameter (in) 2.42		Height (in) 1.00		Water Content	w_o	25.8 %	w_f	19.3 %
Overburden Pressure, p_o	2,000 psf			Void Ratio	e_o	0.76	e_f	0.52
Preconsol. Pressure, p_c	7,000 psf			Saturation	S_o	92 %	S_f	101 %
Compression Ratio, $C_{\epsilon c}$	0.16			Dry Density	γ_d	96 pcf	γ_d	111 pcf
LL	PL		PI		G_s 2.70 (assumed)			

Classification CLAY with SAND (CL), light yellow-brown Source B-1 @ 19 feet

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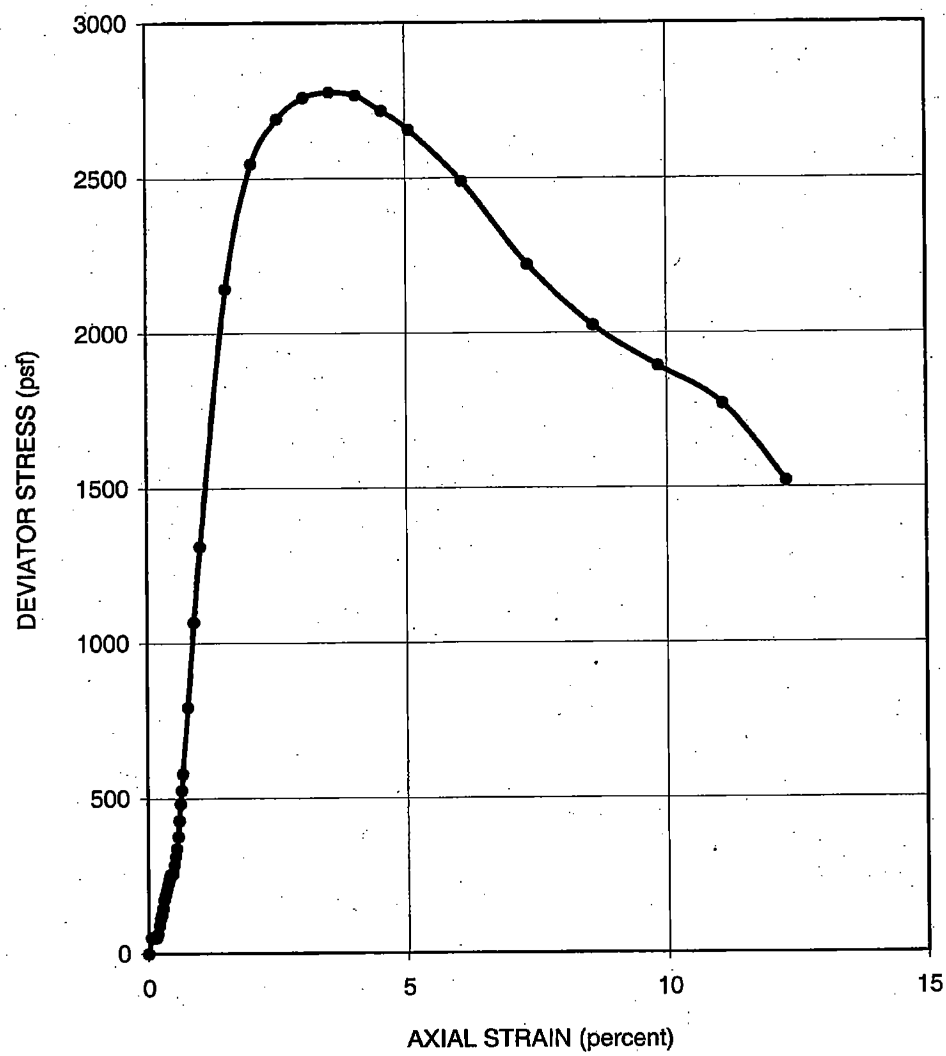
CONSOLIDATION TEST REPORT

Date 06/04/08	Project No. 4783.01	Figure C-1



SAMPLER TYPE	Shelby Tube		SHEAR STRENGTH	1,390	psf
DIAMETER (in.)	2.843	HEIGHT (in.) 5.63	STRAIN AT FAILURE	3.6	%
MOISTURE CONTENT	26.5	%	CONFINING PRESSURE	1,800	psf
DRY DENSITY	99	pcf	STRAIN RATE	0.75	% / min
DESCRIPTION	CLAY with SAND (CL), light yellow-brown		SOURCE	B-1 @ 19 feet	

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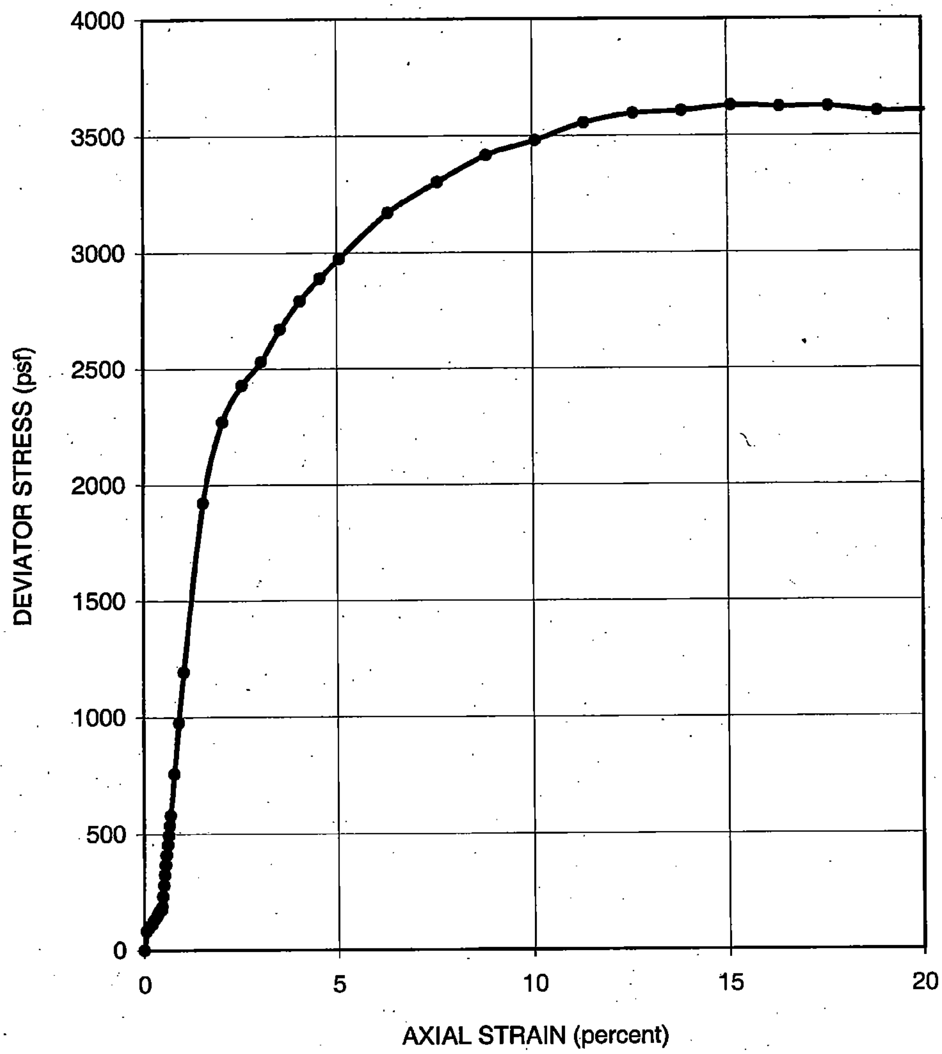
UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

Date	06/04/08	Project No.	4783.01	Figure	C-2



SAMPLER TYPE	Shelby Tube		SHEAR STRENGTH	1,810	psf
DIAMETER (in.) 2.843	HEIGHT (in.) 5.6		STRAIN AT FAILURE	15.1	%
MOISTURE CONTENT	28.1	%	CONFINING PRESSURE	1,800	psf
DRY DENSITY	97	pcf	STRAIN RATE	0.75	% / min
DESCRIPTION	CLAY with SAND (CL), yellow-brown			SOURCE	B-3 @ 19 feet

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UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

Date 06/04/08 | Project No. 4783.01 | Figure C-3

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